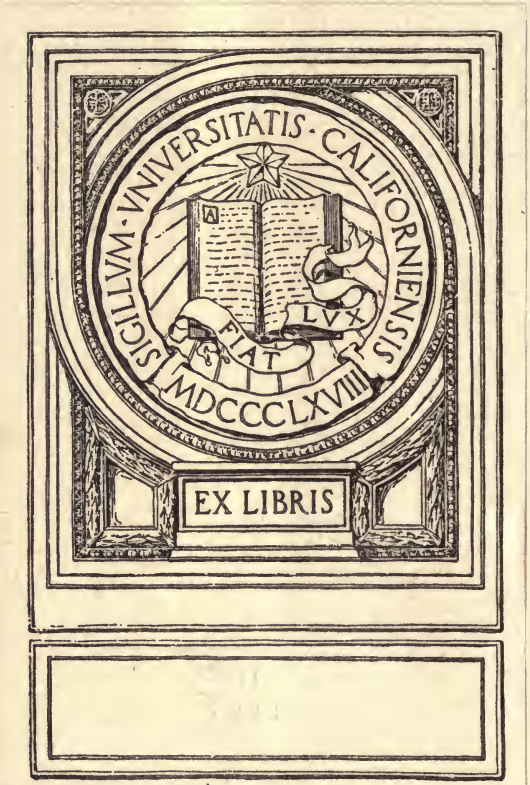
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MASONRY AS APPLIED TO CIVIL ENGINEERING

MASONRY AS APPLIED TO CIVIL ENGINEERING

BEING A PRACTICAL TREATISE ON THE DESIGN AND CONSTRUCTION OF ENGINEERING WORKS IN STONE AND HEAVY CONCRETE

ADAPTED FOR THE USE OF STUDENTS AND THOSE ENGAGED IN THE PRACTICE OF THE PROFESSION

BY

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PREFACE

THE art of masonry is very ancient; in fact, until comparatively recently all structures of any importance were built of stone. Nowadays the advent of steel and ferro-concrete has, not as some would suppose, taken the place of stone for important structures, but has made possible works which, while partly constructed of masonry, could not within reasonable limits be constructed wholly thereof.

As far as stonework and stone-cutting concern the builder and architect, the many treatises on building construction leave nothing to be desired. They treat the subject for the most part, however, from the artistic standpoint. The object of the present work is to consider the question from the point of view of utility, that is heavy works, of use rather than mere ornament. Every endeavour has been made to set forth, broadly, without too much recourse to higher mathematics, the theoretical considerations with regard to structures and members thereof which, while being constructed with the greatest economy, shall resist, beyond doubt, failure or collapse under any normal condition which the designer thinks wise to provide for. These considerations are as far as possible illustrated by practical examples; in fact, this has been the aim of the author right through. It is hoped, therefore, that the work will be useful alike to students and those engaged in practice. Notification of any errors will be greatly appreciated.

F. NOEL TAYLOR

MANCHESTER,
October, 1914

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MASONRY APPLIED TO CIVIL ENGINEERING

CHAPTER I

STONES USED IN CONSTRUCTIONAL WORK

STONE is one of the most useful materials which the civil engineer employs. To the early engineers it was of more importance still, being practically the only material with which structures of any magnitude could be constructed. The selection of stone for any particular work demands great care. The all-important question is that of durability, upon which depends the power of the stone to resist the effects of atmosphere. Primarily this depends upon the chemical composition and physical structure of the stone, the former being important in the case of engineering works in large towns, where any acid in the atmosphere will rapidly attack it. This is principally caused by the rain dissolving the impurities and carrying them into the pores of the stone. Specially detrimental are CO_2, hydrochloric and nitric acids, but stones which are largely composed of magnesium and lime carbonates may be even destroyed by the small percentage of CO_2 in mountain air, while stones containing iron are liable to attack by atmospheric oxygen. Consequently there are certain stones which may stand almost indefinitely in the country and fall to pieces in towns, especially in the latter case would rainfall appear to have a marked influence, because it is principally by rain that the impurities are transmitted. Even the face of a structure fronting the

prevalent winds may suffer more than the other faces, due to the wind, when wet, forcing the rain into the stone, and, when dry, having a wearing effect, although the latter has really only an important bearing in foreign countries where sand is borne along by the wind in large quantities. Stone which is always sheltered from the sun does not get dry after rain, and is liable to decay from this cause if the stone is not of good quality and has a porous surface ; in fact, in engineering work, especially for the face work, no porous stone should be used, and in cold climates frost will also tend to destroy such stone. Some stones are harder than others, but hard stones do not of necessity stand the atmosphere better than soft ones, but in the case of culverts, subject to a continual flow of water, the wear on a soft stone may be considerable.

Judging stones.—It is most important that the engineer should be able to judge a good stone by the appearance and by simple tests. The ability to do the former is not easily or rapidly acquired and demands some practical experience. In the United Kingdom there are certain stones which are known by experience to have the several qualities claimed for them, consequently the engineer need have no hesitation in using them.

It is in works abroad where no such records exist that the importance of selection comes in. The broken surface of any piece of stone may be examined by a powerful glass and should show a bright, clean, and sharp fracture. Generally speaking, the densest and strongest stones will be those which prove the most durable. If the stone has a dull appearance it may decay. Stone in a structure is usually in direct compression and, as this pressure will be considerable in foundations and also in masonry arches, the resistance to crushing is of importance, not so much because the compression on the stones, when in place, is anything like equal to the ultimate compressive strength of the stone, but because it gives a very good idea of the capability of the stone to resist frost. The following table gives a list of the average results of a number of experiments on the crushing strength of stones :—

TABLE I*

	Tons per square inch.	Absorption per cent.
Aberdeen granite	4·84	About ½%
Peterhead	3·65	
Mountsorrel	5·75	
Killiney (Dublin)	4·75	
Whinstone	3·50	—
Penmaenmawr (Graywacke)	7·71	—
Bramley Fell sandstone	2·75	9½%
Craigleith (Edinburgh)	2·41	8%
Mansfield (Notts)	2·25	10·5%
Quartz rock (natural bed)	11·38	—
Compact limestone	3·8	16%
Portland limestone	1·67	13·5%
Purbeck	4·10	17·2%
Bath (Box Tunnel)	0·67	17·0%
Bolsover	3·36	—
Limerick	3·9	—

Absorption.—This is a very important test for stone, as it is a general rule that the best stones are those which take up the least water. It is quite simple to carry out. The stone is carefully weighed wet and dry, the soaking continuing for twenty-four hours. In the above table the second column gives the percentage of absorption.

Weight.—The weight of any particular stone per foot cube will vary, and with it of course varies the density of that stone, and generally in consequence its weathering properties. The table below gives the *mean* weight of several stones, but the actual weight in any particular case is liable to variation as much as $\frac{1}{10}$:—

TABLE II

Stone.	Weight per foot cube.
Granite and syenite	170
Trap and basalt	180
Sandstone	137
Limestone (compact)	162

* These values err on the side of safety.

Stone.	Weight per foot cube.
Limestone (granular)	137
,, (magnesium)	144
Kentish rag limestone	165
Lias limestone	165
Quartz and felspar	145

Chemical Tests.—There is a simple test for stones which is supposed to be an indication of the capability of that stone to resist frost. A boiling solution of concentrated sulphate of soda is prepared and into it are placed small pieces of the stone in question, which, being previously weighed dry, are hung up subsequent to immersion. The salt crystallizes in the stone and causes a certain amount of disintegration, and the stone is then weighed to find out how much has been so detached. The greater the quantity the worse is the quality of the stone from a weathering point of view. Again, if the stone is placed in dilute H_2SO_4 or HCl the acid is considered to have the same effect as a town atmosphere, and in the case of stones having a large proportion of lime and magnesium carbonate, strong acid on the stone will cause effervescence. To ascertain if stone contains earthy or decayed matter a few small chippings (spalls) are placed in a glass of water and allowed to stand for an hour. If the stone is agitated in the water and an earthy deposit or any turbidity results it shows that the stone is not highly crystalline nor are the particles well cemented. The test is useful for magnesian limestones. Now for practical purposes stones fall under four distinct headings, viz. (1) Granites and igneous rocks, (2) Sandstones, (3) Limestones, (4) Slates. The last-named are of minor importance to the engineer. Scientifically stones are known as (*a*) Siliceous, (*b*) Argillaceous, (*c*) Calcareous, having respectively silica, clay, and calcium carbonate as a base.

Granite is the principal stone used for engineering work, and has a crystalline, granular structure, noticeable by most people. True granite consists of quartz, felspar, and mica, the latter being easily discernible in the form of small

flakes which show up like glass in the sunlight, and they can be picked off with a penknife. The colour of the stone depends on the felspar, which should be crystalline and lustrous. The grains are white, grey, pink, red, and brown. There is usually more felspar than quartz, which is a hard, glassy substance appearing as colourless amorphous lumps. Generally speaking, the proportions are 10 per cent of mica, 40 per cent of quartz, and 50 per cent of felspar, but they are liable to much variation in different stones. The quantity of felspar is important, because upon it depends the durability of the stone, although it makes it harder to work. Both potash felspar and lime or soda felspar are found in granite, sometimes both in the same stone. They are known respectively as orthoclase and oligoclase, the former being the first to decay. Mica is always liable to decay, and is a source of weakness in the stone.

An excess of lime, iron, or soda, especially when the iron occurs as *marcasite*, are all detrimental to its durability and strength. Large dark-coloured iron stains may often be seen in granite, and they are a good indication that the stone is not fit for good work, facings at any rate. There is a variety of granite known as syenitic granite (or syenite) in which the mica is replaced by hornblende in the case of true syenite, but in syenitic granite there is both hornblende and mica. Granite which contains hornblende is usually darker in colour than the others and the stone is generally tough and good for general work.

Granite is quarried by wedging and blasting, heavy blocks for engineering work being, as a rule, obtained without powder, as it tends to break up the stone. The density of granite makes it very much in demand for all classes of engineering work, and it will wear well and resist the action of water and air. Kingstown Harbour near Dublin is almost entirely constructed of Killiney granite, a special railway having been laid down to the quarries some three miles distant. Granite may be had from various parts of the kingdom, the Aberdeen granite being perhaps the best known, but good granite is had all over Scotland, including Peterhead, Ross of Mull, Stirling Hill, and Dalbeattie.

Leicestershire produces a very hard granite containing a good deal of hornblende. The principal firms in the granite trade are :—

Atlas Stone Co., Ltd.
British Macadams, Ltd.
Brookes, Ltd.
Charnwood Granite Co.
Clee Hill Granite Co., Ltd.
Cliffe Hill Granite Co., Ltd.
Croft Granite Co., Ltd.
Ellis & Everard.
Empire Stone Co., Ltd.
Enderby & Stoney Stanton Co., Ltd.
Field & Mackay, Ltd.
Forest Rock Co., Ltd.
Freakley, J. & Co., Ltd.
Griffiths, W., & Co.
Groby Granite Co., Ltd.
London Granite Co., Ltd.
Mountsorrel Granite Co.,Ltd.
Northern Granite Quarries, Ltd.
Ord & Maddison.
Penmaenmawr & Welsh Granite Co., Ltd.
Potts, J., & Son.
Pwllheli Granite Co., Ltd.
Read & Son.
Rowley Regis Granite Quarries, Ltd.
Shap Granite Co., Ltd.
Threlkeld Granite Co., Ltd.
Tytherington Stone Co.
Wettern Bros., Ltd.
White, J. R., & Co.

Cornish granite is somewhat soft, having an excess of mica, and is liable to weather badly owing to an excess of potash felspar, which turns into kaolin (china clay).

Irish granite, especially that from the counties Dublin and Wicklow, is very useful for all classes of engineering work and is much in evidence on the Irish railways. Felstone porphyry, quartziferous porphyry, gneiss, mica slate (or mica schist), and hornblende slate are igneous rocks, but they are not true granite and, except where found locally, are not used for the best engineering work.

Trap rock, or whinstone, which is a mixture of felspar and hornblende, is much used for road metalling and setts, but not for structural work, except for rubble and concrete.

Sandstones.—The harder varieties of sandstone are used by the engineer. Sandstone essentially consists of grains of quartz cemented together by silica. The cementing material may, however, be iron oxide, carbonate of lime or

magnesia, or even alumina, while the quartz may be replaced in part by mica limestone and soft clayey or limy matter, which are usually looked upon as impurities. This is because the quartz may be considered in itself as imperishable. The durability of the stone primarily depends on the nature of the cementing matrix. There is also a variety of sandstone (considered by some as a siliceous limestone) in which the substance is mainly carbonate of lime and the matrix silica.

For engineering work those stones which have a coarser-grained substance, and known as *grit* stones, are most useful. They are quarried in the millstone grit formation and are heavy and can be had in large blocks.

Liver rock is a Scotch term for a dense freestone which can be quarried in large blocks and has no natural cleavage, such as may be seen in York stone, so largely used for flags owing to the ease with which such stone may be split along the lines of cleavage, which are really very thin beds or plates of mica. The colour of sandstone varies very much from nearly black to almost white, but the colour is due to the presence of iron in some form or another. Blue or grey is caused by carbonate of iron, and in some cases by phosphate of iron; red colour by anhydrous sesquioxide; brown or yellow by hydrated sesquioxide.

There is a variety of sandstone known as metamorphic sandstone. It has been subjected to heat at an early period and is generally too hard for working, but may be used for rubble. One of the most important sandstones for constructional work is the Bramley Fell stone. This stone is found near Leeds, and although the original quarry is long since worked out, the stone still bears the name and to a great extent the qualities of the original stone. It is a millstone grit stone and can be had in large blocks. Its weathering qualities are sometimes impaired by the presence of too much potash felspar. The weight per foot cube of sandstone is important, because when the weight falls below 130 lbs. per foot cube the quality of the stone is inferior. On placing chips of the stone in water and stirring about as before stated the resulting turbidity

of the water should be very small. Nor should the absorption of the stone greatly exceed 5 per cent of its weight after twenty-four hours' immersion. The effervescence in acid should be small. The principal grit stones are found in Yorkshire near the coal measures, and a few in the new red sandstone formation, while from Halifax and Bradford come the celebrated York stone paving slats. In Gloucestershire there is found the well-known Forest of Dean stone, also near coal measures in the mining district of Lydney. Both hard and soft stone are found, the former being well suited to engineering works and can be had in blocks of large dimensions. This stone has been used extensively in the docks at Newport, Swansea, and Cardiff. The Nottingham stone, while a very excellent sandstone, is of more importance to the architect than the engineer. In composition it is a siliceous dolomite of the Permian formation lying between the new red sandstone and carboniferous series. There is also a white stone of this variety. For facing work they are excellent. The façade of St. Pancras Station is of red Mansfield stone.

Limestone.—A stone is said to be a limestone when its essential constituent is carbonate of lime. There is a large variety of this most important stone : heavy stones contain a large proportion of carbonate of magnesia and are known as dolomites. Those which contain clay are useless for any structural work. Other stones contain iron and silica. Generally speaking, the carbonate of lime in limestone makes it unfit for exposure to town atmosphere, owing to its liability to attack from dissolved CO_2, but this is not always the case, as a great deal depends upon the texture of the particular stone. Those which are hard and dense, of a fine grain and homogeneous structure, are the best to withstand the effects of the atmosphere. For external work limestone is valuable in the very uniform colour which this stone has and the ease with which it can be worked, especially when freshly quarried, this stone having the peculiar property of hardening on exposure to the atmosphere, due, it is stated, to a slight chemical change in which some of the softer particles are removed

during the time that the quarry sap is dried out. The most important variety is the Portland stone, which is obtained in the Isle of Portland (Dorsetshire) from the upper oölite series of sedimentary rocks. The stone is classified according to its depth in the quarry, and, working downwards, we have, (1) true roach, (2) white bed, (3) curf,* and (4) base bed. The true roach is blasted and the others are removed by wedges and levers. The chemical analysis of all these stones is practically the same and may be taken on an average :—

Carbonate of lime . . .	95·16 %
Magnesium carbonate . . .	1·20 %
Silica	1·20 %
Dark alumina	·50 %
Water	1·94 %

The weight per cubic foot is 134 to 140 lbs. and the colour may vary from bluish white to light brown. The true roach is the best stone, and is a peculiar one, containing as it does the screw fossil in large numbers. The stone is somewhat liable to cavities, but for marine and hydraulic works is very valuable, as it stands water well. The white bed is principally used for architectural work and the curf and base bed for internal work. Kentish rag is a limestone variety, but so far as structural work is concerned it is only used for rubble. Limestone may be considered as compact or granular. Compact limestone consists of either pure carbonate of lime or the same in combination with sand and clay. It has a dull appearance and a dark blue-grey colour. Kentish rag is a compact limestone and also lias limestone, but the latter is used principally for the production of mortar. It contains clay. Granular limestones are those principally used for structural work. They come from the oölitic formation and consist of grains of various sizes and fossil shells. The colour is light yellow and light brown. They are, however, absorbent and do not stand the acid-laden air of towns. They can, however, be had in large blocks, which should be laid on their natural bed. The stone is comparatively light, rarely more than

* Called Bastard Roach.

140 lbs. per foot cube. Portland stone comes under this classification.

Magnesian limestone, such as Mansfield yellow stone and Bolsover Moor, are those stones which have a tolerably large proportion of carbonate of magnesia (over 15 per cent and usually about 40 per cent), together with 4 or 5 per cent of silica. When the proportions of carbonate of lime and magnesia are 1 to 1 the stone is called a dolomite, in which the peculiar form of crystallization is of value to the stone. The result is a particularly homogeneous stone, such as the Bolsover Moor, in which the magnesium and lime are, so to speak, intimately blended together and not separately discernible under a magnifying glass. Magnesian limestone is, however, liable to the attack of H_2SO_4 in the atmosphere. In the Chelmark stone, produced from the Portland series and used formerly in Westminster Abbey, the cementing matrix is of a siliceous nature, which adds greatly to the weather-resisting properties of the stone.

Preservation of Stone.—Of late years attempts have been made to treat stone in such a way that it will resist the effects of the acid-laden atmosphere of our towns. The various processes have only met with a small measure of success, principally owing to initial first cost. Primarily they aim at filling the pores of the stone, the solutions themselves being organic substances in a soluble state. Other solutions are to be had which act in a different way, namely, by acting on the constituents of the stone to which they are applied so as to form a soluble compound to fill the pores and harden the surface of the stone. It may, in fact, be described as a sort of " Case-hardening " process applied to the stone. Another point against the practical use of these preservatives is that for really good effect the stone should be subject to a vacuum before treatment, which, when large blocks are in question, adds seriously to the cost of plant and treatment. Oils have been tried. They are more or less effective in keeping out acid and moisture but discolour the stone so much that their use is rarely met with. *Soluble silica* when used will combine

with the constituents of the stone and render the surface hard, filling up the pores meanwhile. It has the disadvantage of causing subsequent efflorescence on the face of the stone. Flint may be made soluble in caustic soda under pressure for this purpose. Caustic soda is an alkali, and in the form of solution is caused to combine with the lime in the form of limestone, with the result that the alkali separates from the silica and forms a hard, insoluble silicate of lime, which remains in the pores of the stone and sets hard.

Ransome's system is well known, and consists of coating with silicate of soda and chloride of calcium, the surface of the stone being previously well cleaned and dried. The blocks, when dressed and cleaned, are painted with a 1 to 3 solution of the silicate until they are saturated, and any excess is then dried off; when the silicate is quite dry the chloride of calcium is applied with a brush. The two solutions cause a double decomposition. The silicate of lime which is formed being insoluble fills the pores of the stone and hardens it, and makes it very much less liable to attack by the atmosphere. Four gallons of each solution will under ordinary circumstances coat 100 super. yards of surface. St. George's Hall, Liverpool, and Trinity College, Dublin, are among the principal buildings to which it has been applied with success. Coating the surface of the stone with silicate of potash or silicate of soda is known as Kullman's process. The silicate of potash will decompose, for instance, on limestone, for which it is very suitable, and silicic carbonate of lime and silica will be deposited. It is liable to discolour the stone.

CHAPTER II

LABOUR ON STONES AND GENERAL REMARKS ON MASONRY

THE working of stone into suitable shapes for constructional work is a very ancient craft and, as may be seen from very old structures, was carried out by the Ancients in no mean way; in fact, it is still a matter for wonder how such works were carried out under conditions so very primitive as compared with those at the present day. On the subject of quarrying it is not proposed to make any remarks here, as, except on very large works, the actual felling of the stone does not usually come under the jurisdiction of the engineer, and also that it is a complete study in itself and upon which text-books are to be had. Suffice to say that when quarried in the shape of blocks the work of dressing the stone for use commences. Good masons are not always obtainable, as the trade is one which has a good standing. Good foremen-masons are men who usually have a great deal of practical knowledge which is useful to the engineer, and, in passing, it may be said that the engineer will do well not to interfere too much with these people. Many of our ablest engineers were originally masons, notably Telford, and it is surprising to find how little present-day engineers understand about the practical side of the subject, and those who have the opportunity would do well to acquaint themselves with it.

Tools.—Various tools are employed for working stones, and these are shown in Fig. 1. They are mostly used for cutting the stone. For dressing the stone in the various ways shown later on other tools are used, notably the scabbling pick, which has two pointed heads, the serrated pick, which has two serrated heads, the punch, and the axe, and the patent axe. Various forms of hammers are used by masons, generally the scabbling hammer, spalling hammer, the waller's

hammer, and mash hammer. Some of these are shown in Fig. 1. Granite is dressed by heavy picks and axes after being roughly shaped with the scabbling hammer. The chisels are used for cutting mouldings, rebates, etc.

The tools used by masons for cutting stone, being for the most part of simple form, are made from hexagonal chisel steel, which is a market form.* On large works these tools are made and sharpened in the smith's shop and the

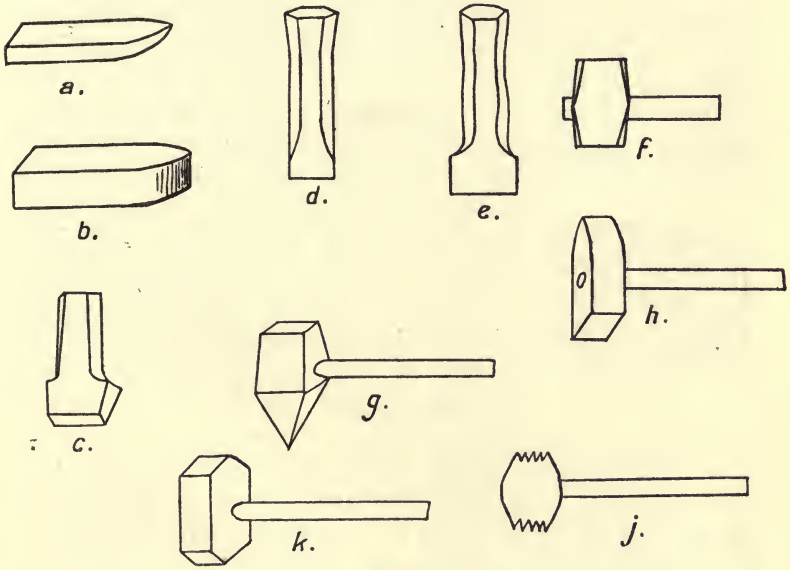


Fig. 1.

a. Gab. *b.* Wedge. *c.* Pitching tool. *d.* Inch tool. *e.* Boaster. *f.* Mash hammer. *g.* Scabbling hammer. *h.* Waller's hammer. *j.* Serrated pick. *k.* Axe.

engineer should see that the best steel is obtained for the purpose.

Tempering.—The tempering of masons' tools can be done by any intelligent smith. It consists of placing the cutting end in the fire till bright red and then suddenly quenching it in cold water. It must then, while still hot, be immediately withdrawn and rubbed at the end with a piece of soft stone. The point of the tool rises in temperature due to the heat still retained in the body of the steel

* Preferably Landore "H" or "SH" brands.

and a film of colour (due to oxidation) appears on the polished surface. Various colours appear, and by experience it can be judged by the colour (which is an indication of the temperature of the steel) when it must be finally dipped in the water and held there till cool, moving it about in the water meanwhile. The tool is plunged at the appearance of various colours, according to its subsequent use, and for stone-working tools the correct colour may be described, as near as possible, as yellow tinged with purple, but experience is necessary before the colours can really be appreciated. They vary from very pale straw-yellow to pale greenish blue.

Faces of Stones.—In engineering work, to which we are confined rather than architectural, the faces to which the stones are brought are not usually of an elaborate nature, owing to the expense of dressing, such faces being quite unnecessary. Boldness of face is much to be preferred owing to the solid appearance of the whole work resulting. The hammer-faced work shown in Fig. 2, sometimes called hammer-blocked work, is done with a scabbling or spalling hammer. If stones are squared and simply left rough from the hammer for the quoins or face of a wall they are termed hammer-faced ashlars. The term *ashlar* in masonry is one which has been held to have various meanings, but is generally held to mean stones that are carefully worked and are usually over 12 ins. in depth, but need not be confined to stones cut to a plain square face. Stones squared up and under 12 ins. in depth are called *shoddies*. Hammer-dressed work may be brought to a fair face with the scabbling pick, and is called scabbled or roughly picked work. A better face still may be obtained by means of the same tool, and is known as finely picked work, and sometimes as dabbed or daubed work. All these surfaces are similar and vary in smoothness from the plain rock-face as it comes from the quarry to the finely picked. For engineers' work there is not much gained by cutting the fair faces. By

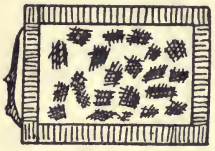


Fig. 2.

means of the axe a smooth face may be obtained, and such work as quoin's rebates is treated in this way. With the patent axe the finest of all surfaces except polished work may be had. The face of this tool is formed of a number of parallel thin steel blades, clamped together and capable of removal for sharpening. In good work such as piers and abutments of bridges the appearance of the work is much enhanced by drafted margins. The joints may also be more rigidly inspected by the engineer. In fact, it is usual to run a draft or smooth surface an inch or more in breadth round the margins of squared stones in order to ensure close-fitting joints. The drafted margins, as shown in Fig. 2, are worked with the axe, as in single or fine axing, or the margins may be chisel drafted. The other principal faces used for masonry are pointed work, shown in Fig. 3, boasted work, shown in Fig. 4, which consists of making a number of parallel chisel marks across the surface of the stone by means of a chisel having an edge about $2\frac{1}{2}$ ins. wide, called a *boaster*. It will be seen that the chisel marks are not kept in con-

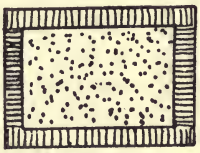


Fig. 3.

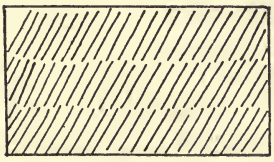


Fig. 4.

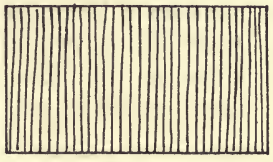


Fig. 5.

tinuous rows across the whole width of the stone. They may, however, be kept in continuous lines, as shown by Fig. 5, in which case the work is termed tooled work. The effect of the labours shown in Figs. 4 and 5 is to increase the effect of large plain surfaces by adding a number of shadows and high lights.

Cutting Stone.—Sandstone, when fresh from the quarry, may be cut with a saw, and the face so left is very smooth and is known as sawn work. With hard stones,

such as granite, the blocks have to be split. This is done by drilling holes by means of a chisel with a bulged end, shown in Fig. 6, the gab or wedge shown in Fig. 1 being used to split the stone, or, in the case of very large blocks, the plug and feathers are used, as shown in Fig. 7, so as to prevent the edges of the stone splitting. The holes originally made in cutting the stone may be observed in any important engineering structure. Large blocks are dressed square by chiselling a "sinking" at each corner of the block, into which small pegs are inserted, made of hard wood about 2 ins. square. Across each two adjacent cubes straight-edges are laid. By the eye in line with the under side of one straight-edge and at right angles to it the other may be sighted through, and if not true can be made so by adjusting the sinkings until the bottoms of the straight-edges are out of winding. When this has been done straight drafts are worked from one sinking to another along opposite edges until there is a draft all round the outside. The superfluous stone is then pointed off and dressed to a finished face in parallel drafts.

Fig. 6.

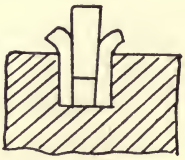


Fig. 7.

In working a block of stone, one face, usually one of those which will subsequently be the bed of the stone, is brought to a plane surface, and upon which is set out the required shape with the square or templet; chisel drafts are then sunk across the ends of one of the adjacent faces by means of the square or level, according to the shape, and the second face is worked between these drafts. Subsequently the position of the third face is ascertained, and so on till the cutting is complete. To form a plane surface the stone has a series of drafts cut as shown in Fig. 8. The superfluous stone between these drafts is knocked off till the

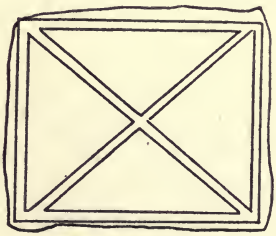


Fig. 8.

whole is fair to the straight-edge, two of which of equal depth are used, and across which the eye can ascertain if the surface at all points is a plane one. If the stone has to be worked to a winding surface one of the straight-edges has one of its edges out of parallel with the other, so that its depth is variable according to the amount of wind and the distance apart at which they will be used. The straight-edges are sunk into drafts at either end of the stone till their top edges are parallel. If the stone is cut away between and to the depth of each draft the surface will have the requisite wind. The cutting of stone to a winding bed is applied to such cases as cutting the stones for skew arches, etc., which will be fully treated later on. The twist has to be very accurately cut.

Fig. 9.

The rule which has its edges out of parallel is called a *winding-strip*, and its dimensions have to be ascertained for the work in question. The development of spirals is a subject which belongs to geometry, with which the reader should and is assumed to be well acquainted. For instance, suppose in Fig. 9 *a b* is the development of any spiral, Fig. 10 is assumed to be the right section of the quarter cylinder from which *a b* is the developed spiral. Then, if the arc *e f* is made

Fig. 11.

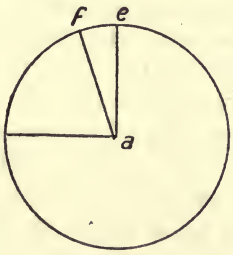


Fig. 10.

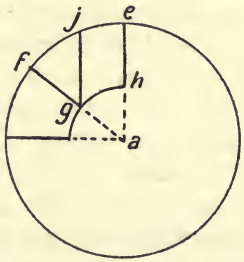


Fig. 12.

equal to *a c* and *f a* joined, the sector *a e f* will represent the winding-strip, or rather the triangular portion of it, because it must be remembered that it will consist of two portions, as shown in Fig. 11, the plain portion being

equal and corresponding to the straight-edge and the shaded, triangular portion being equal to *a e f* in Fig. 10. Twisting-rules, when applied to a stone, must be kept in parallel planes at a distance equal to that of the cylinder from which the spiral is developed, and to do this they are usually kept at the required distance by light iron stays. Twisting-rules are made quite thin, and the working edges are round and *not* square. The distance between the twisting-rules should be small, or else in the above construction an error may arise, because it is really only an approximation, although sufficiently accurate for practical purposes. It has also been assumed in the foregoing that the width of the spiral surface is equal to the radius of the circumscribing cylinder. If it is less the construction must be modified. The development of the spiral is the same as in Fig. 9, but in the case of Fig. 12, join *f a*, and from *g*, the point in which *f a* cuts *g h*, draw *g j* parallel to *e h* and *f g j* = the triangular portion of the winding-strip. The rest of the procedure is the same as before. The winding-strips *may* be set out as considering it a portion of a spiral heading plane, and is often done by operative masons, but is not accurate, and should be discountenanced on good work.

So much for the plane surfaces, the labour on which is termed plain work and is generally placed on bed and side joints of stones in construction. Circular work is employed to a limited extent in engineering, and is principally applied to bull-nose coping-stones and stone mooring-posts on docks, etc. The work does not call for any special attention; circular columns in architecture are usually executed in a lathe. If, however, any of the sides of a block of stone must be formed into conical, spiral, or spherical surfaces the matter becomes more complicated, and the stone has to be brought to a series of plane or cylindrical surfaces, on to which the templets for finishing the work are applied.

These preparatory surfaces are termed the *surfaces of operation*, and large blocks are usually worked roughly to such surfaces at the quarry. Surfaces of operation are

generally made vertical or horizontal, so that the lines for the bevels and templets may be directly taken from the plan and section. The term circular work is applied to the labour put upon the surface of any convex prismatic body, such as a mooring-post or stone column. If the work is concave, such as the under side of an arch voussoir, it is called *circular sunk* work. In the formation of a ball the labour is termed *circular circular* work, and if in the shape of a basin, such as shown in Fig. 13, it is *circular circular sunk* work. The two latter forms of labour rarely apply to engineering. Mouldings are sometimes required on external work, and in this connection are generally worked by hand. The profile of the moulding is cut in a zinc templet and marked from this on the two ends of the stone to be treated. A draft is then sunk in the two ends to the shape of the required profile and the stone cut away and gauged with the zinc templet. The cutting of arch stones, and stones for curved walls, etc., will be discussed when we come to consider each of these items. In the meantime attention is drawn to Figs. 14 and 15 and the following paragraphs :—

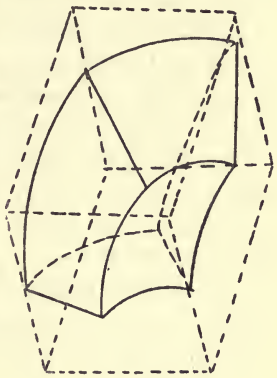
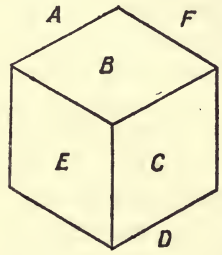
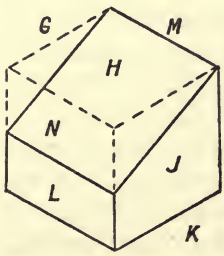


Fig. 13.





Figs. 14 and 15.

"It is necessary that the student of engineering should have some knowledge of the labour on stonework. When a stone is sawed, the surface left by the saw is known as *half-sawing*, because half the cost is charged to each part of the separate pieces. A stone which comes straight from the quarry is said to be *self-faced*. The term also applies to a stone split in two, but the more irregular

angles of a stone are chipped off at the quarry, a process known as *scabbling*. A piece of stone sawed or brought to a fairly good surface with a chisel is said to be *half-plain* work ; those accurately done, that is, with the surfaces truly out of winding, are known as *plain work*.

"Stones are often worked by rubbing them together, water and sand being added to assist the process, which is often done by machinery. It is known as *rubbed* work. It rarely applies to engineering.

"A frequent point of difficulty to observant students is the following, of which an explanation is given.

"Why do not sunk and moulded work include labour to preparatory faces ?

"In endeavouring to show why the labour to half-bed should not be deducted where sunk work is measured, it will be simpler first to find out why it should be deducted where plain face is measured, and then ascertain in what manner the conditions of sunk or moulded work vary from plain work. Imagine a block of stone to be converted into a cube, and consider the labours on it. Half-sawing would be necessary to all six faces A, B, C, D, E, F, but it would only be measured to faces having no other finish, namely, the back F, because it is covered by subsequent labours on all the other faces. Half-bed would be measured to top and bottom faces B and D, and half-joint to ends A and C. Plain face would be measured to front E. Thus all faces have been measured. Now if plain face is required to the whole or part of any of the other faces, it is clear that to make it equal in value to face E the preparatory labour (whether bed, joint, or sawing) previously measured must be deducted, because they are identical with and form part of the actual labour to plain face. Now imagine another cube prepared similarly to the previous one, namely, half-sawing to back M, half-bed to H and K, half-joint to G and J, and plain face to L, and assume that a splayed face N is formed from the back edge of H, terminating half-way down L. After the preparatory labours are worked to H, K, G, J, the surface H is destroyed and chipped away or sunk to the new plane of N and finished as a plain face.

The finished area of N is measured as sunk plain face (or sunk face), and the remaining part of L (in this case one-half) as plain face. Half-bed to H and half-joint to parts of G and J are not deducted, because it is necessary that they should be prepared before the sinking can be properly set out, and where the sinking is deeper than 3 ins. a dimension of rough-sunk preparatory labour must be taken, and this further indicates that no deduction of bed or joint can be taken. The same applies to mouldings. Summing up these results, we find that half-beds, half-joints, and half-sawing (otherwise the preparatory labours) are deducted where plain face is measured over them because they form part of the plain face labour; but they are not deducted where plain or moulded work is sunk from them, because they have to be worked first, then destroyed, and a new surface formed in the required plane, so that they form no part of the subsequent labour." *

The manipulation of large stones is a matter which demands attention, and nowadays construction is greatly expedited by the employment of labour-saving appliances, it being remembered that of all *power* man-power is the most expensive. For raising moderately large stones some form of tackle such as Weston's blocks are employed. They may be slung from a tripod or beam or other place. The top block contains two sheaves of different diameter, cast together so as to form one piece, the runs of each being formed so as to receive the links of the hoisting chain and prevent it slipping. The bottom block contains one sheave only. The chain is endless, and passes first round the larger of the top sheaves, and then the bottom sheave, and finally the smaller top one. The two ends are connected and allowed to hang loose. In raising a load the loose chain passing from the larger top sheave is pulled, and when lowering, the other. It is not necessary to go into the mechanical principles of this most useful appliance, but the mechanical advantage is very large, that is, a small pull of the chain gives a very much greater pull upward of the hook, which, however, rises

* *Civil Engineering Practice*, by the author, Charles Griffin and Sons.

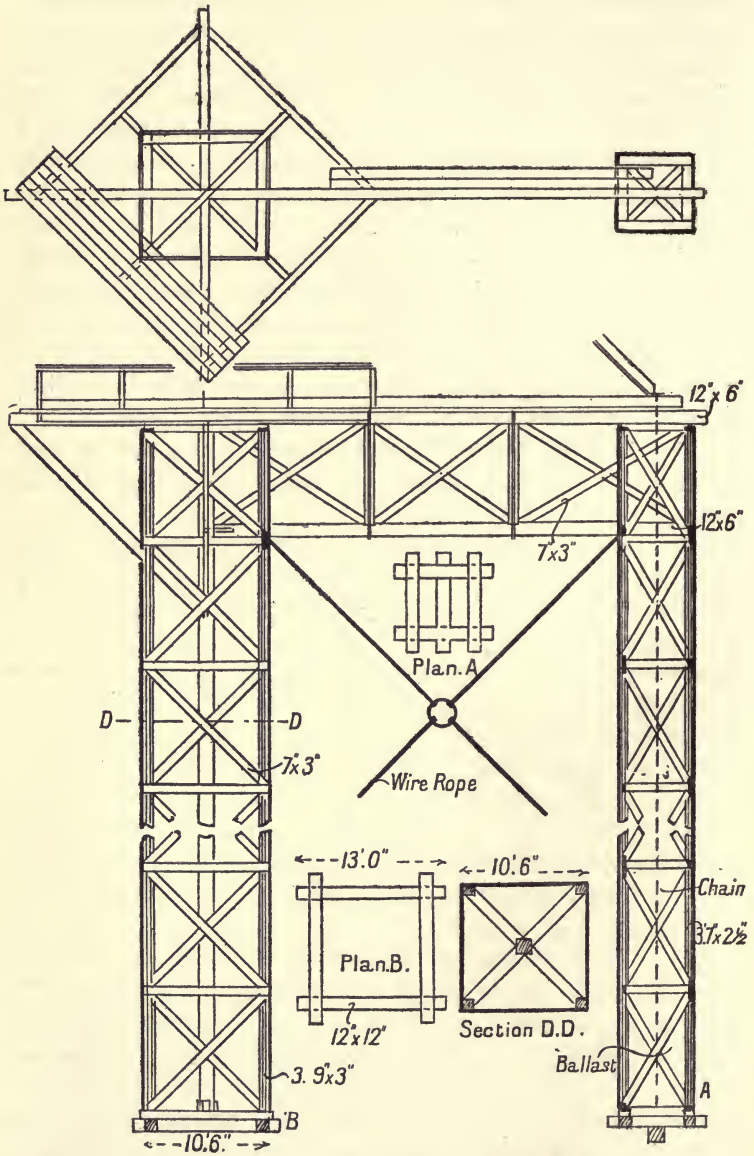

12"x 6"
12"x6"
7"x3"
Plan. A
D
D
7"x3"
Wire Rope
13'.0"
10'.6"
Chain
3"x2½"
Plan.B.
12"x12"
Section D.D.
Ballast
3. 9"x3"
A
B
10'.6"

Fig. 16.

very slowly. The load will remain stationary at any point.

If R and r=radii of the two top sheaves and P=pull applied, then for any value of P the load raised is equal to

$$W = P \times \frac{2R}{R - r}$$

or if

R=10 and r=9 and P=100 $W = 100 \frac{2 \times 10}{10 - 9} = 100 \times \frac{20}{1} =$ 2000 lbs.;

but for every 20 ft. of chain pulled the load will only rise 1 ft. The ordinary locomotive travelling crane is, of course, always conspicuous on large works, but on land the derrick and gantry crane are largely used. They are very useful where a supply of electricity can be had. The design of cranes is a subject which, of course, cannot be treated in a book of this sort, but some information on the subject is given in the appendix to the author's *Manual of Civil Engineering Practice*. The derrick crane may be said to consist of two parts, the staging and the jib, together with the machinery necessary for lifting. The jib is designed according to the maximum load and radius of that load on principles enumerated in the treatise named. They are usually of steel, consisting of four angles laced by light steel bars so [ ] having an increased cross-section in the centre. This and the machinery is placed on a staging which may be of considerable height. Often they are made of timber, on the lines shown in Fig. 16, the scantlings given being suitable for a crane up to $3\frac{1}{2}$ tons capacity. The most important point is to provide sufficient ballast at and above A, which usually consists of bricks piled up

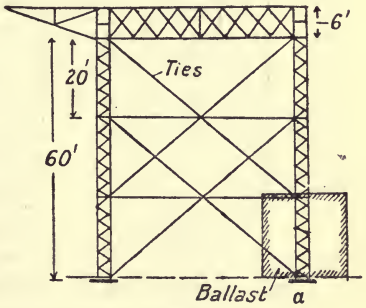


Fig. 17.

inside the timber post, and down which passes a chain as shown, to take the load. The pull on this chain, which must be always less than the weight of the ballast, may be found by a simple graphic construction and will be apparent to the student of engineering. This chain has been one of the points which have proved a disadvantage to the use of timber stagings, and it is now usual to make these cranes of steel. While more expensive in first cost, the cost of removal and re-erection on different works or part of the same work is a good deal less, although this is not always appreciated; and in the case of timber stagings it is often found cheaper to sell the old stuff, which fetches a very small price, owing to the sections having been notched and bored, and the use of new timber means all the work over again. The carriage charges for steel are about two-thirds that of timber, and the cost of erection is low enough to allow of the staging being taken longer distances, so saving the expense of making new stages, as is the case when timber is used. The initial cost of a steel stage is 30 per cent more than a timber one, but re-erection is 60 per cent less. It is impossible to discuss the question of design here, but the diagrams in Figs. 17 and 18 will be suggestive to the designer of the general arrangement. The legs would be built in sections of say 20 ft. long and in cross-section composed of four angles and light lattice bracing. They would be all interchangeable, so that the top girder could be erected at any height. The braces and stays should be all made interchangeable. The legs should have a light steel grillage attached to the base, upon which would be piled the ballast all round the legs. With derrick cranes wire ropes are usually employed for lifting; at the end of this rope or chain as the case may be is some form of crane-hook, particulars of which are given in the *Manual of Civil Engineering* on page 781 before referred to. Attached to this must be some means of gripping the

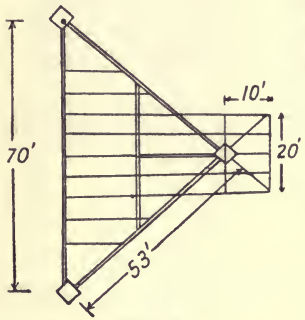


Fig. 18.

stone. This may be done in a variety of ways, and in the case of small blocks is simply a chain slung round the stone and tied in such a way that it will not slip. For large stones it is usual to cut a slot in the face of the stone for the insertion of a Lewis, which is shown in Fig. 19, the construction of which will be at once apparent. It consists in all of six pieces, including the key, and is taken asunder when the stone is in place. This is obviously impossible in the case of marine works, for which the modified form shown in Fig. 20 is used, the small chain actuating the releasing wedge. Soft stone may be raised by lifting-tongs, shown in Fig. 21, the idea of which is quite plain. In Fig. 22 is shown another form of Lewis. It has no loose parts and the mortise is curved and cut deeper than the plug, so that when the plug is down at the bottom the key can be inserted, while the lifting of the plug puts it all tight. It is most important that the engineer should see that all the lifting appliances, cranes, etc., are in first-class order, as he will probably be looked to for an explanation of an accident.

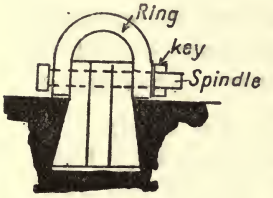


Fig. 19.

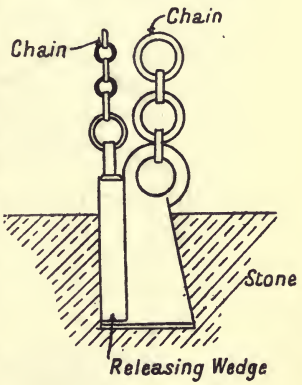


Fig. 20.

Plant.—Upon efficient means of transport of heavy materials many works of heavy masonry are dependent for their construction. In many large works, such as Kingstown Harbour near Dublin, light railways have been laid down specially for the purpose, but only in exceptional cases will the expenditure of such a proceeding be warranted. For moderate-sized works, such as reservoir dams, the wire ropeway presents at once a cheap and efficient means of transport, in fact may, in the case of rough and undeveloped country, be the only means whereby materials and even men can be brought onto the works. So important

is the subject that a few words may not be out of place here, although they must be taken as suggestive only, the reader being referred for full information elsewhere. The wire ropeway was used for transport on the Derwent Valley dams and the Loch Arklet (Glasgow Waterworks) dam, and elsewhere. There are various systems, each of which are specially adaptable to special cases. They are :—

1. Endless rope to which the carriers are fixed.

2. Fixed ropes along which the carrier is drawn. There may be one or two fixed ropes ; in the latter case the carrier goes up one and down the other. An endless rope provides the motive power. In the case of the material being

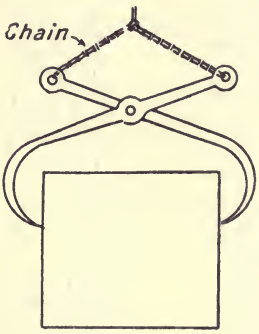


Fig. 21.

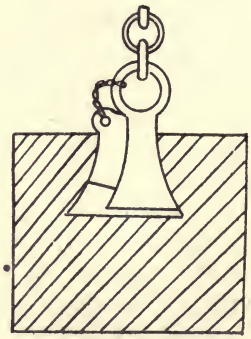


Fig. 22.

quarried at a higher elevation than the works the endless rope may be dispensed with and the load run down by gravity. The endless rope system is confined to places where the gradient will not be more than 1 in 3 and the load not more than 500 tons per hour, in single loads of less than 6 cwts. It is not suited to spans of over 600 ft. All this applies to cases where the carrier is not fixed but simply relies on gravity to hold it to the rope ; where it is fixed by clips the whole conditions are changed, and we have a system which can be used with the steepest gradients and sharpest changes of level. Its one disadvantage is the unhooking of the carriers for loading unless the rope runs very slowly so that they may be filled in motion. The speed could not well be over two and a half miles per hour. They

are automatically tipped for emptying. For heavy loads the fixed rope must be employed, and this is generally necessary for engineering works. Its first cost (as a temporary measure) is somewhat against it, but in many places its adoption will mean economy in the end. Single fixed ropes are generally employed, and loads of 5 tons may be transported. The rope may be slung at an angle of 45° and spans up to 6000 ft. may be operated. The rate of transport is slow, but usually as fast and faster than the stone transported can be quarried. In the construction of a ropeway there is really nothing which cannot be designed by the ordinary engineer. The supports or trestles are either steel or timber, suitably braced to withstand the load and rigidly bolted down to firm foundations. Angles or tees generally form the four sides of the support. They terminate in a tee at the top, and are spaced at about half the distance apart at the base. Steel is much better than timber, and although more in first cost can be sold when the works are complete. The steel rope carrying the load hangs in the form of a catenary, and the strain due to the load is treated accordingly and will be found discussed in the author's *Civil Engineering Practice* on page 114 and elsewhere.

Referring to the Loch Arklet scheme mentioned above, the following particulars from *The Engineer* may be of interest:—

"We understand that with carts and horses it was estimated that the cost of transport would work out at about five shillings per ton, while with the ropeway this figure would be reduced to nearly half. The line of ropeway as constructed under this scheme has a length of about 7000 ft., with a total rise of 525 ft. The nature of the ground may be judged from the fact that in places near the shore the rope has an inclination of 1 in 1½. After rising until a point about a mile inland has been reached, the line becomes horizontal, and then dips down gently to the terminus at the dam.

"The loads transported in this way are made up of cast-iron pipes, gravel, sand, cement, timber, stones, pieces of machinery, etc. In the case of loose material, such as

sand, cement, etc., iron skips, having a length of 6 ft., a width of 4 ft., and a depth of 1 ft., are employed on the line. These skips are suspended by a flexible connection attached to a cross-beam running between the two hangers of the pulley carriages. By employing two pulley carriages the load is prevented from twisting round, while the flexible connection ensures that the skip shall always be horizontal even on the steepest portions of the incline. For transporting timber, pipes, and like articles, the load is attached directly by chains to the cross-beams. The average net load carried by each car is about 25 cwt., so that, as twenty cars can do the journey per hour, the capacity of the line is 25 tons. This, on occasion, can be increased to 50 tons per hour.

"Although it might appear that the work and expense of erecting such a ropeway could hardly be justified by the limited time for which it will be in use, it should be borne in mind that, quite apart from the reduced cost of transport already mentioned, the contractors will incur no expense for the upkeep of roads, and will be able to carry on work in snowy and frosty weather—a matter of importance in view of the climate and character of the district. It can be well understood that the erection of the ropeway was in itself quite an undertaking. Manual labour alone was available for hauling the materials required for it up the steep slopes. Concreting was used for the foundations of the pillars, and at several points where projecting rocks would have interfered with the passage of the skips blasting operations had to be resorted to. The plant was supplied by Adolf Bleichert and Company, of Leipzig, and was erected by the contractors for the Loch Arklet works."

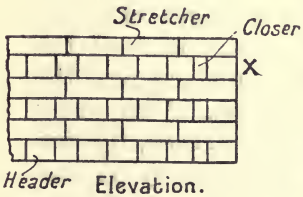


Fig. 23.

Bond.—Stones, like bricks, are laid to a certain bond, although not usually so regular. That is to say, each stone must overlap the ones above and below it to ensure broken joints. No particular arrangement is adhered to in masonry, although English bond, Fig. 23, viz. alternate headers and

stretchers in the same course, is generally found. For heavy work, such as dock walls, etc., block in course is employed, shown in Fig. 24. The stones are all squared and brought to

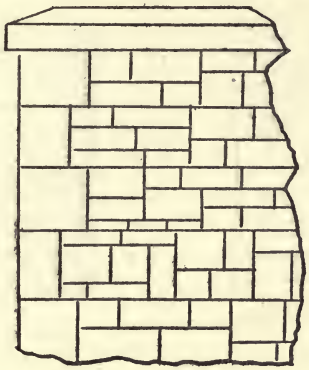


Fig. 24.

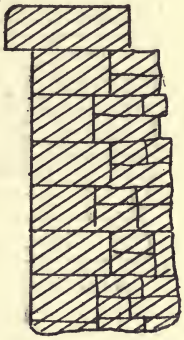


Fig. 25.

good, fair joints and the faces hammer-dressed. Ashlar is only used as a facing to rubble or other backing, shown in Figs. 25 and 26 with drafted margins. Squared coursed rubble is largely employed, as shown in Fig. 27, while if it is squared

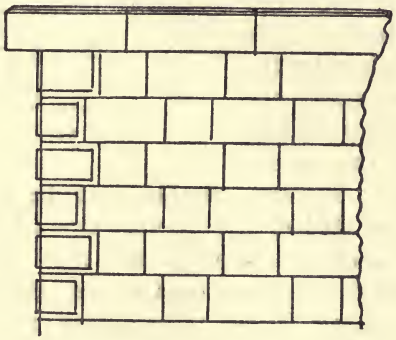


Fig. 26.

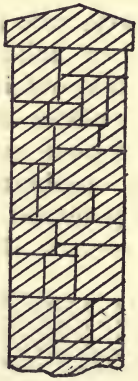


Fig. 27.

and not regularly coursed it appears as in Fig. 28, in which there are several small headers, called snecks. Of course, there are very many other ways of building masonry walls.*

* The term *wall* in masonry construction has a very wide significance, and may include anything from a dry stone fence to a reservoir dam, and is here used in a general sense.

In architectural work a variety of patterns on face may be worked according to the arrangement of stones, but engineering work usually resolves itself into an ashlar or

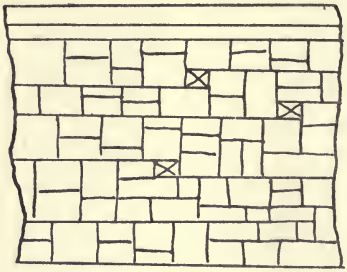


Fig. 28.

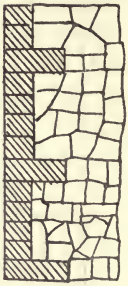


Fig. 29.

coursed rubble facing and a rubble backing. A coursed rubble wall may be lined with brickwork, as shown in Fig. 29. Random rubble is the term given to walling built of stones which are not squared.

In the case of uncoursed rubble set in mortar the stones are used as they come from the quarry, care being taken that they are as uniform as possible. The bond is obtained by fitting in the inequalities of the stone, using one bond stone every super. yard on face, any openings between the stones being pinned by spalls. Random rubble may be built in courses of 12 ins. to 18 ins. deep, every stone being bedded in mortar so that there shall be no continuous vertical joints. It is a common practice to bed only the face stones and grout the others. The practice is bad and should not be countenanced in good work.

Fig. 30.

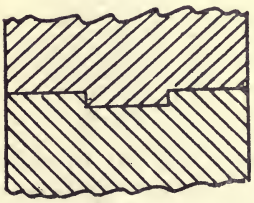


Fig. 31.

Coping-stones.—For bridge parapets, the tops of retaining walls, etc., some form of stone is used which fulfils a twofold object, that of

giving a finish to the work and of protecting that work from an undue amount of wet. The simplest and most generally employed is the saddle-back coping shown in Fig. 30, which is a large stone projecting 3 ins. or so each side of the wall. The throating is an important point. It serves to prevent water creeping down by capillary attraction on to the face of the wall. A similar cope stone may be made with single sloping face, in which case there is only one throating, or it may be segmental on top. It may also take various fancy forms in the case of bridges, the consideration of which concerns the architect. Cope stones of dock and sea walls are usually bull noses or bevels.

Fig. 32.

Joints.—The jointing of stones in heavy masonry is much more than simply dabbing on a piece of mortar and bedding the stone in it. In the case of marine works the joints of the stones are often subject to a good deal of tension which an ordinary mortar joint will not stand, and measures have to be taken to resist this tension. The subject will be referred to again when sea walls are considered. For resisting lateral displacement the table joint and the joggle are used, the construction of which is shown in Figs. 31 and 32. For resisting movement in all directions the metal cramp shown in Fig. 33 is used, principally for coping-stones. The cramps are thin metal bars of a size and sectional area according to the nature of the work. The ends are turned downwards and cut rough with a chisel. Mortises of dovetail shape are cut and the cramp is run up with lead. Bronze cramps are used in very good work. They are sometimes set in cement or sulphur and sand.

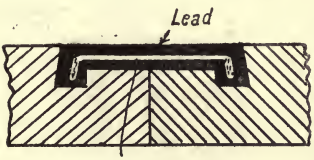


Fig. 33.

Fig. 34.

In Fig. 34 is shown the lead plug joint which is used for coping-stones. Slate dowels of the form shown in Fig. 35 are also used for the same purpose. For ashlar work liable to the force of waves the lead plug is employed. It is shown in Fig. 36. Stones may also be cut with V grooves, which when the stones abut appear so ◇ in plan. The space is filled with cement and sand, 2 to 1.

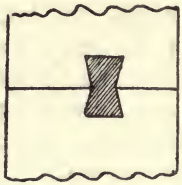


Fig. 35.

For attaching ironwork to masonry two methods are employed. The first is shown in Fig. 37 and is known as a rag bolt. The bolt has its end flattened and bulged and cut to a rough surface with a chisel. It is bedded in a dove-tail slot and run up with lead. The system is used for

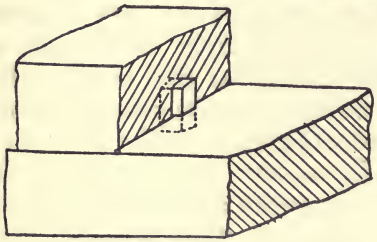


Fig. 36.

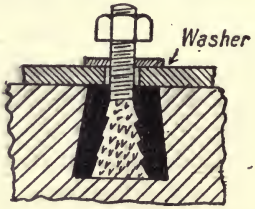


Fig. 37.

attaching shackles, etc., on quay walls and mooring-rings, but for iron standards (such as railings on sea walls) lead is often used, in the way shown in Fig. 38. Such standards may have a square, flat base and four rag bolts. The cutting of the four holes, however, tends to weaken the stone, and the writer has found that those run with lead are not so liable to fracture round the base in heavy seas.

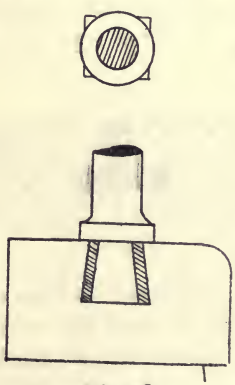


Fig. 38.

Mortar may be of two kinds, cement mortar and lime mortar, but in good work cement mortar is invariably employed nowadays. The subject of cement is dealt with in many books

and cannot be fully treated here. It is sufficient to say that only the very best English, artificial Portland cement of a well-known brand should be used. It should come up to the following specification :—*

The cement shall be of the very best quality, and subject in every respect to the approval of the surveyor.

It shall be packed in strong, perfectly sound sacks. Any torn or hard sacks will not be accepted.

The cement shall be subject to the following tests :—

It shall be very finely ground, so that not more than 5 per cent by weight shall be stopped by a sieve with 76 by 76 meshes per square inch ; it shall have a specific gravity of not less than 3·1. Samples made into briquettes and immersed in water twenty-four hours after being made, and remaining immersed until tested, shall have an average tensile strain, seven days after being made, of not less than 350 lbs. to the square inch ; and twenty-eight days after being made, the average tensile strain of the briquettes then tested shall be at least 20 per cent more than the average of the seven days' test. The cement shall not set in less than one hour, and shall be considered set as soon as no impression can be made on it with the thumb-nail. Other tests may be applied at the surveyor's discretion.

Pats of cement made with water, and allowed to set for five hours in a damp atmosphere, at a temperature of 100° F., and afterwards immersed for nineteen hours in water at the same temperature, shall show no deterioration.

If the cement does not come up to these tests, it shall be removed from the store by the contractor, and at his own expense, within fourteen days of written notice to that effect being given to him by the surveyor.

Samples for testing to be taken from the consignments, at will, by the surveyor or his representative, who shall have the tests made, the cost for doing which shall be defrayed by the contractor.

For building strong brickwork and stonework, and for making it waterproof, mortar made of Portland cement

* *Vide* also the British standard specification.

and sand is used. For laying bricks or for grouting* the sand should be comparatively fine, while for concrete or coarse mortar it should range from fine to coarse.

The usual proportions for mortar are 1 part Portland cement to 3 sand, but sometimes 1 to 2 is used for stronger work, and 1 to 4 for weaker work in brick or masonry.

If a large quantity of sand is added to the Portland cement the mortar will be "short" and brittle so that it will not work well. For many kinds of work a quickly hardening mortar is required, of no great strength, and if the mixture only worked well, 1 of Portland cement to 5 of sand would be sufficient; but, as such a mortar would be short and adhere imperfectly to the stone or brick, the difficulty is overcome by mixing slaked or hydraulic lime with the Portland cement to correct the faults of a mixture poor in Portland cement. Such a mortar is cheap and very satisfactory; it hardens rapidly, possesses marked hydraulic properties, and obtains great strength on exposure to air.

The following proportions by volume are suitable :—

1 Portland cement,		5 sand,	½ lime paste.
1 ,,	,,	6 to 7 sand,	1 ,,
1 ,,	,,	8 ,,	1½ ,,
1 ,,	,,	10 ,,	2 ,,

Lime Mortar.

Lime mortar is used in some places, such as where good lime can be made on the works by burning limestone, and is quite satisfactory if the time of setting is not important and if no great strength is required. The sand used for mortar must be sharp and very well washed, pit sand being the most suitable; but river and sea sand have often to be used, and they must be specially well washed. The form of Joint usually applied to masonry work is shown in Fig. 39 and is called a struck joint. Roman cement has very little strength and is not used much for jointing masonry. It sets very rapidly.

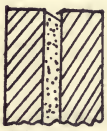


Fig. 39.

* Grouting will be considered under the heading of Underpinning.

Lime.

Lime is made by the calcination of limestone, which consists very largely of carbonate of lime ($CaCO_3$). During the process of calcination the calcium carbonate is converted into calcium oxide (CaO) by the removal of carbon dioxide (CO_2), which is driven off by the heat in the form of a gas. The following equation represents the decomposition of limestone by heat, $CaCO_3 = CaO + CO_2$ (carbonate of lime = quicklime + carbon dioxide). On mixing with water the quicklime becomes converted into calcium hydrate by combination with the water, thus $CaO + H_2O = Ca(OH)_2$ (quicklime + water = calcium hydrate). This slaking reaction is accompanied by a considerable development of heat and a great increase in bulk. The process of setting consists in the absorption of carbon dioxide from the air. $Ca(OH)_2 + CO_2 = CaCO_3$ (calcium carbonate) $+ H_2O$, thus the setting of pure lime is practically the reversal of the processes of calcining and slaking. If the lime is mixed with any other material, such as sand, the process of setting is not so simple, it being complicated by the formation of silicates, etc. It is these extraneous substances which complicate the setting and hardening of cements, and render the slaking process so difficult that the cements mostly have to be ground to powders, and which also cause the setting to follow so quickly after the slaking has commenced that in some cements no period for slaking need be allowed. This is the case with Portland cement. For a further consideration of this subject, such a book as Munby's *Introduction to the Chemistry and Physics of Building Materials*, published by Constable and Co., may be consulted. The proportions of lime and sand in mortars for different purposes depend upon the quality of the mortar. The purer the lime the greater the proportion of sand that can be mixed with it, but on the other hand, no great strength is attained by pure limes, especially when used in thick walls. A pure lime may have sand mixed with it in the proportion of 3 parts of sand to 1 of lime, and such a mortar would be the poorest that should be used for any purpose. An hydraulic lime, such

as greystone lime, should have about 1 to 1½ parts of sand to 1 of lime ; while a lime such as lias lime should have only an equal proportion of sand mixed with it.

Cohesive Strength of Joints.

In the case of marine works the cohesive strength of mortar is an important question, and the experiments of Mr. Frank Latham, engineer to the Penzance Corporation and Harbour Board, as recorded in Vol. XXXII of the *Trans. of I.C.E. in Ireland,* are of value. In reference to the cohesive strength of cementing materials Mr. Latham says:—

"For testing this, experiments have been made by the author by means of an attachment of his own fitted to an Adie machine. The attachment is now being manufactured by Mr. Patrick Adie, and supplied along with his briquette-testing machine. The beam is slightly increased in weight by a small addition required for balancing the new attachment, which is slightly heavier than the ordinary briquette holder. The dog-grips and rod are easily detached when the machine is required for briquette testing. For a cohesive test, several pieces of granite, or other stone intended to be used in the work to be carried out, are roughly squared on the sides, and shaped in pairs, with their corresponding faces either left as they came from the quarry or roughed for special tests. A pair of 'dabs,' or dog-holes, should be made in the sides of each stone for the grips to lay hold in. The two stones in each pair are then cemented together, with either neat cement mortar, or concrete, and after twenty-four hours' setting are immersed in water for a given period. When ready for testing, the pair of stones are removed from the water, the under stone is secured in the vice-jaw, which is fixed in the floor, and the upper stone is fastened between the teeth of the dog-grips, which are connected with the testing machine by the adjustable straight steel tension rod. The adjustment and testing are carried out in precisely the same manner as in briquette breaking."

Some of Mr. Latham's results are tabulated below :—

7 days' immersion,	18 lbs.,	result of	3 tests.
,, ,,	30 lbs.	,,	1 test.
,, ,,	22 lbs.	,,	1 ,,
Average	21 lbs.		
6 days' immersion,	34 lbs.,	result of	1 test.
,, ,,	25 lbs.	,,	1 ,,
,, ,,	19½ lbs.	,,	1 ,,
Average	26 lbs.		

Series 11.—Mortar of 1 measure cement and 1 sand—

49 days' immersion,	32 lbs.,	result of	1 test.
,, ,,	42 lbs.	,,	1 ,,
Average	37 lbs.		
21 days' immersion,	17 lbs.,	result of	1 test.
,, ,,	40 lbs.	,,	3 tests.
,, ,,	20 lbs.	,,	2 ,,
,, ,,	35 lbs.	,,	5 ,,
,, ,,	27 lbs.	,,	1 test.
Average	32 lbs.		
14 days' immersion,	25 lbs.,	result of	1 test.
,, ,,	27 lbs.	,,	3 tests.
Average	26½ lbs.		

General Remarks on the Execution of Masonry

The following conditions are laid down by Professor Rankine and cannot well be improved on :—

" 1. To build masonry as far as possible in a series of courses, perpendicular or as nearly perpendicular as possible to the direction of the pressure which they have to bear, and to avoid all long continuous joints parallel to that pressure by ' breaking joint.'

" 2. To use the largest stones for the foundation course.

" 3. To lay all stones which consist of layers, or ' beds,'

in such a manner that the principal pressure which they have to bear shall act in a direction perpendicular or as nearly perpendicular as possible to the direction of the layers. This is called laying the stone on its natural bed, and is of primary importance to strength and durability.

"4. To moisten the surface of dry and porous stones before bedding them, in order that the mortar shall not be dried too fast and reduced to powder by the stone absorbing its moisture.

"5. To fill every part of every joint and all spaces between the stones with mortar, taking care at the same time that such spaces shall be as small as possible."

Joints.

In the case of blocks of stone, especially the softer stones, such as sandstone and granular limestone, the length must be limited to a multiple of the depth or else the stone will be liable to fracture. This is generally three, and in the case of granite and hard stones four or five times the depth. All bed joints should be dressed to plane surfaces. If this is not done properly the joints will either be open or, what is worse, *flushed* (not flush as sometimes stated by students), as shown in Fig. 40, in which case the whole load comes on a very small piece of stone and is apt to splinter it. This kind of joint will find its way into masonry by

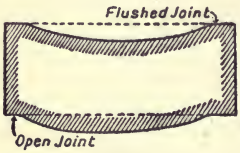


Fig. 40.

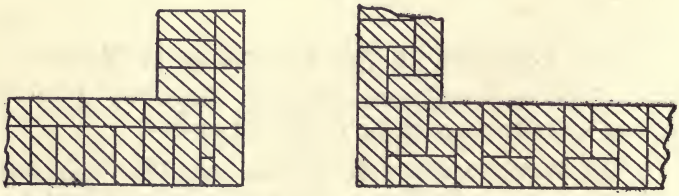


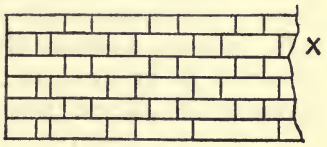
Fig. 41.

bad workmen, who seek to misrepresent their work, and careful inspection alone will prevent it. While they must be plane, smooth surfaces are not essential to a good joint,

but all stones should be fitted in place dry before jointing. The joints are usually broken in one of the two ways shown in Fig. 41. In no case should the headers form less than one-quarter the total superficial area, and one-third is to be advised as a stipulation in specifications. True English bond gives more than one-third. The quoins should be specially selected, and the larger the better. The joints in good masonry should not exceed $\frac{1}{8}$ in., and quarry-faced or hammer-dressed stones should always have chisel-drafted margins. In the case of coursed rubble masonry there should be a number of bond stones. They should show on face an area of not less than $2\frac{1}{2}$ sq. ft. in 1 sq. yd. They must be the entire depth of the course and of a breadth = twice the depth. They should be roughly squared and the beds hammer-dressed. Bond stones must not be placed in successive courses above each other. Between these stones the work is built up of smaller ones, sometimes two or more to the depth of the course.

Rubble Masonry.

Such stones may taper in width but never in depth. Coursed rubble masonry demands very rigid inspection on the part of the engineer to see that all the foregoing rules are conformed with, or else there will be a great many spaces simply filled with mortar instead of small stones bedded properly as they should be. One cube yard of good masonry should not take more than $5\frac{1}{2}$ cub. ft. of mortar. Coursed rubble masonry is only used for backing ashlar and block-in-course, and its resistance to crushing per unit area is less than one-half that of a single block of stone of the same area. Rubble may also be built in random courses, that is, the course may be of different depths and at various levels, but each course must be a plane bed in itself. Uncoursed rubble masonry is not used in engineering work, and is principally confined to fence



Elevation.
Fig. 42.

walls. In the case of ashlar and block-in-course backed with rubble, bond stones, dressed fair on the faces and on the beds which are included in the facing, are used. They may be rough where they tail into the rubble, on the back and sides, with the upper and lower beds roughly dressed. It is very important, too, that both the facing and backing be raised at the same time, or else the bond will be defective. Ashlar or block-in-course backed with rubble is, of course, cheaper than where the whole wall is one of the first-named, and in the case of retaining-walls is just as efficient, because the pressure, as will be shown in the next chapter, is concentrated towards the face of such walls; and although this does not theoretically apply in the case of bridge abutments, coursed rubble will generally be found strong enough to resist the pressure which is concentrated at the back of the abutments, and is generally found used in such places when faced with ashlar for the sake of appearance and protection from the weather. Masonry is generally measured in cube yards, so far as engineering work is concerned. In the case of architectural work, and by some engineers who prefer the method, the *rod* or *rood* is used and is generally held to denote 36 sq. yds. of masonry of a specified thickness. The superficial measure is taken in roods to estimate the cost of the face work, and then the area in superficial roods of the face of each portion of the work is multiplied by the ratio which its thickness bears to 2 ft., so as to compute the cubic contents in solid roods of 36 sq. yds. in area and 2 ft. in thickness in order to estimate the cost of the masonry exclusive of the face.

CHAPTER III

RETAINING-WALLS AND EARTH PRESSURES

OF works in engineering masonry, that of retaining-walls is one of the most important. Many works when analysed simply resolve themselves into retaining-walls, but in this chapter our remarks will be principally confined to those walls which resist earth pressure, and those which sustain water will be treated in the next chapter. The whole subject of retaining-walls aims at fulfilling one main object, that is to provide a wall whose weight shall act in such a way that its moment shall be greater than the moment set up by the earth pressure behind the wall in an opposite direction, and the design may be made by graphical construction. In the case of roads and where building materials of sufficient size are handy retaining-walls may be built of dry stone, which is at once a saving in expense, and in the case of roads which bring forth no monetary return from their use this is an important consideration. Flat-bedded stones are best suited to this purpose. Again, dry stone walls are self-draining, and in consequence serve a useful purpose in this direction in the case of roads. They would be built with a batter of 1 to 8 to 1 to 12. Usually, however, retaining walls in engineering work are of heavy masonry set in cement mortar. In the first place we must consider the strength and weight of materials used in walls, as shown in the following table :—

TABLE III.

Material.	Safe dead Load. Tons per sq. ft.	Factor of Safety.	Weight, lbs. per ft. cube. *
Granite masonry . .	15 tons	20	160
Portland and hard limestone .	15 ,,	15	140
Sandstone	12 ,,	10	130
Blue brick (in cement) .	9 ,,	8	120
Stock brick ,, .	6 ,,	5	115
,, Lias mortar .	5 ,,	4	112
,, Grey ,, .	3 ,,	4	112
Portland cement concrete .	5 ,,	8	130
Lias lime concrete . .	3 ,,	8	120
Compact earth . . .	2 ,,	—	112
Made ground . . .	1 ,,	—	100

And also the properties of earths shown as follows :—

TABLE IV

Nature of Soil.	Angle of Slope.		Weight per Cubic Foot.	Value of $\tan \frac{a}{2}$.
			lbs.	
Gravel	40°	Angle of repose	112	·488
Dry sand . . .	38°		92	·554
Sand	22°		116	—
Vegetable earth . .	28°		83	·414
Compact ,, . .	50°		112	·466
Shingle	39°		141	·512
Rubble	45°		153	—
Clay (well drained) . .	45°		112	·767
Wet clay	16°		124	·677
Water	90°	Slope with vertical	62·5	1·00
Excavated earth . .	56°		79	·532
Gravel and sand . .	52°		115	·488

Knowing these values, then, we can proceed by a general formula to find the mean thickness of any proposed wall and to set out on paper its proposed cross-section.

Now given—

D = height of proposed wall in feet,

W = weight of a cubic foot of the wall (*vide* Table), lbs.,

w = ,, ,, earth at back (ditto), lbs., and

T = thickness of wall in feet also (mean thickness),

$$T = \cdot 7D \tan \frac{a}{2} \sqrt{\frac{w}{W}} \quad . \quad . \quad . \quad (1)$$

Tan $\frac{a}{2}$ signifies that we take the tangent of half the angle of

* These weights are approximate only, and are liable to variation in different cases.

slope. This, of course, varies. It is about ·414 for ordinary earth and ·767 for stiff clay. In practical calculations, however, we may be safe in taking it as ·50. If, however, we are designing a wall for water, we must take it as 1·0, and also remember that water weighs 62·5 lbs. per cube foot in practice. These remarks apply to walls which have the backing trimmed off quite flush with the top of the wall and level. In many cases, however, it slopes upwards from the top—in fact, usually so. This, of course, will alter our figures by increasing the load. They are then known as *surcharged revetments*.

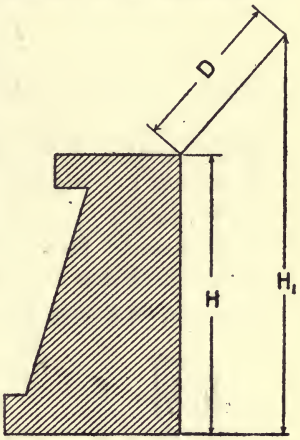


Fig. 43.

To make use of Equation 1 (see Fig. 43), set off D=H and substitute H for H_1 as scaled off. When, however, the earth falls over the top of the wall to the front edge we do not account for it, but we measure D from the point X, as shown by the same figure.

Buttresses and Counterforts.

Long retaining-walls should have counterforts every 20 ft. or so. They are not so much use in giving extra strength to the wall as they are in holding the earth backing together.

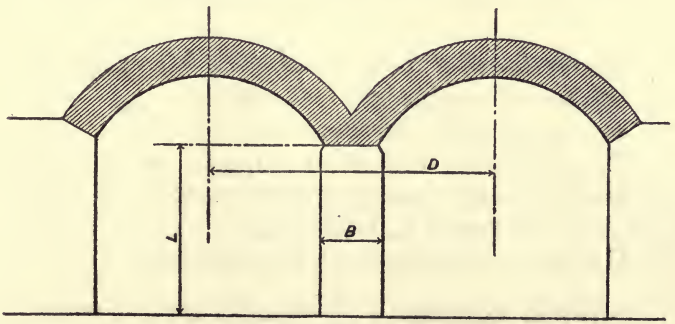


Fig. 44.

For economical and other reasons, retaining-walls are sometimes built with a series of buttresses which are connected by arches, as in Fig. 44. For the design of such walls we employ two formulæ. Using the lettering on the plan—

$$L = T \times \sqrt{\frac{D}{B}} \quad . \quad . \quad . \quad . \quad (2)$$

and

$$B = D \times \frac{T^2}{L^2} \quad . \quad . \quad . \quad . \quad (3)$$

while T=the thickness of the wall, would be found by Equation 1. A particularly good form of retaining-wall is shown in Fig. 45 with a front batter. The thickness would, of course, be found by Equation 1, T being taken as the mean thickness at the middle; we would then draw it out with the required batter, say 1 in 10 (in any case it should never be more than 1 in 6), and make the back stepped as shown.

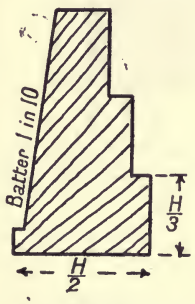


Fig. 45.

The use of the formulæ which we have enumerated is resorted to when we want to design a retaining-wall for a particular purpose, insomuch as we can draft out an *approximate* form of cross-section, but they are only capable of general, not special application. The more usual way is, when a sketch design has been made, to find how the wall satisfies the principles of moments as applied to the stability of walls. It is this which we are now going to discuss.

In retaining-walls the moment of stability is proportional to—

1. The heaviness of the material (=W in equations).
2. The breadth and height (=b and h ,,).
3. The square of the thickness (=t^2 ,,).
4. The inclination of the bed joints to the horizon.
5. The distance of the centre of resistance from the centre of gravity of the wall (horizontally).
6. The angle of slope of the material (=a).

Now say in Equation 1 we found a certain value of T. This value applies to walls having a plumb back. When we

have a batter matters change slightly. Thus, taking the straight wall as unity=1·00, for a batter of 1 in 12 on the face it becomes 0·86, a batter of 1 in 8, 0·80, 1 in 6=0·74, and 1 in 5=0·72. But if the internal face is stepped, its value is 0·85 when the face is vertical.

Graphical Methods.

So much, then, for formulæ. We proceed to investigate a simple graphical method of procedure which may be adopted to meet cases of earth retaining-walls. The reader must refer to Fig. 46.

G D is what is termed the line of rupture, and B D G is the wedge of earth which we consider as providing active pressure on the wall. Considering 1 ft. length of the wall, calculate the weight of this wedge of earth at 112 lbs. per foot cube. Set off K D=$\frac{1}{3}$ D B, and to the scale of lbs. adopted set off H K=weight of this wedge of earth, making K H J=the angle of repose, viz. 30°, produced to meet a line drawn from K perpendicular to B D. J K will then denote to scale the horizontal thrust on B D due to the wedge of earth. Now find the centre of gravity of the wall. As the wall is not rectangular in cross-section, the following construction will be useful for this purpose. On A B produced on either side set off x, x equal to C D, and likewise on C D produced set of y, y=A B. Draw the diagonal lines as shown, and G, the intersection, is the centre of gravity of

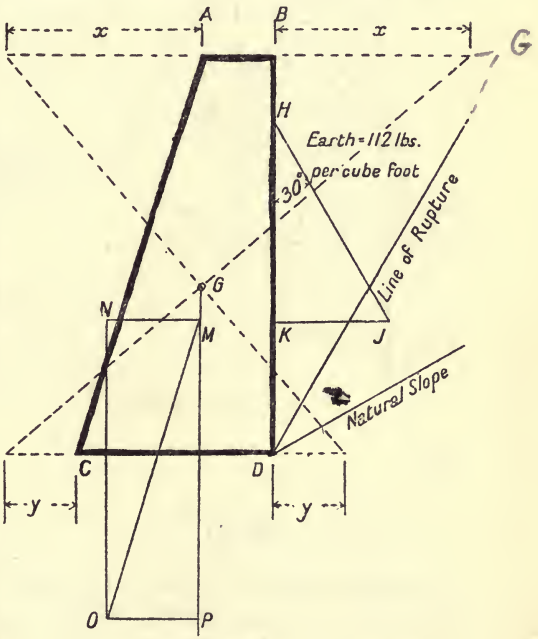


Fig. 46.

the wall. From it drop a perpendicular, G P, making M P the weight of the wall at 130 lbs. per foot cube, in this case=3920 lbs. Complete the parallelogram and draw M O. If, then, M O passes within the middle third of the base of the wall it is safe.

In the case of a wall such as Fig. 45, having a stepped back, the above construction may be used if a line is drawn

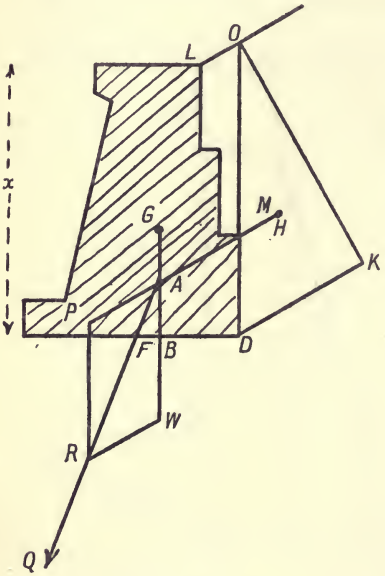


Fig. 47.

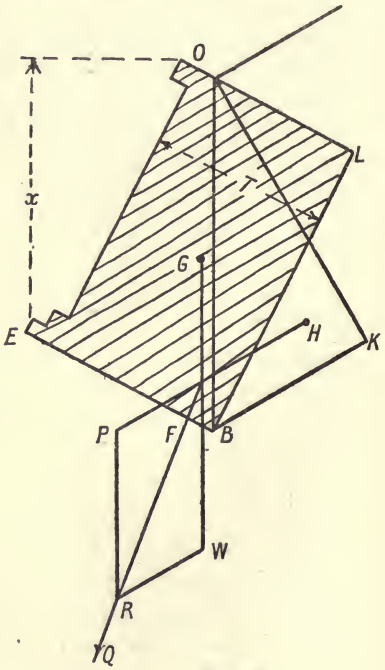


Fig. 48.

from the top to the bottom corner of the back of the wall. This line is then treated as the vertical back of the wall. A more accurate method would, however, be the procedure shown in Fig. 47.

A W represents to scale the weight of the actual wall over a run of, say, 1 ft. *plus* the weight of earth which would be contained in the irregular space M O L. But in Fig. 48, B O L is considered as equal to that in masonry, concrete, or other material, *less* such a quantity of earth. The pressure A P

is always parallel to the surface of the bank, which is, of course, the natural slope of the ground, and is equal to the weight of the earth prism O D K.

This quantity multiplied by the perpendicular distance from F to P H will give us the overthrowing moment of the wall=M. The reason that some retaining-walls have a slanting base is to reduce the obliquity of pressure, and to allow the condition of frictional stability being fulfilled.

Pressure and Tension at Base.

Having drawn the diagram for any retaining-wall it remains for us to find how the pressure at the toe of the face of the wall and the tension at the base of the back of the wall are going to work out. The first must be within the safe crushing strength of the stone, and the latter a very small quantity indeed. With the diagram drawn very accurately, measure along the base of the wall a distance=d from centre of wall to the point where the resultant cuts the base, which, of course, must be within the middle third of that wall. Call it d feet. Let W=weight of wall in lbs., tons, etc., per foot run. A=area of base of wall 1 ft. wide. $Z=\frac{1}{6}BD^2$ where B=width and D the thickness of the wall (B is usually 1·0 as 1 ft. in width of the wall is taken). While a fourth quantity=M=W$\times d$ is required.

Then important we have a familiar equation—

$$\frac{W}{A}\pm\frac{M}{Z} \quad . \quad . \quad . \quad . \quad (4)$$

This important equation may be reasoned out as follows :—

Let a member of a structure be subjected to a longitudinal thrust=W, and be also supporting a load which produces a bending moment M at any chosen section.

Let I=moment of inertia of the section;

y=distance of either extreme fibre of cross-section from the neutral axis (which passes through the centre of gravity of the section);

p=unit stress at the extreme fibre produced by the bending moment;

then it is proved in any book on structural mechanics that

$$p=\frac{M}{I}y \quad . \quad . \quad . \quad . \quad (5)$$

If p_1=unit stress produced by W,

$$p_1=\frac{W}{A} \quad . \quad . \quad . \quad . \quad (6)$$

where A=cross-section area.

Therefore total stress at the given point in section

$$=\frac{W}{A}\pm\frac{M}{I}y \quad . \quad . \quad . \quad (7)$$

The + sign applies when M produces compression at the given point, the minus sign when tension. Many text-books denote $\frac{I}{y}$ by the symbol Z, and term Z "the section modulus," since for any chosen section it is a constant.

It must be remembered that there will be two values for Z, say Z_c and Z_t, the first for the side of the neutral axis in which compression is produced by bending and the other for the tension side, and so

$$Z_c=\frac{I}{y_c} \text{ and } Z_t=\frac{I}{y_t} \quad . \quad . \quad . \quad (8)$$

where y_c and y_t are the distances of the extreme fibres on the compression and tensile sides of the section. In symmetrical figures, foundations, this correction will, of course, not be necessary.

The addition and subtraction of the two sides of the equation gives the compression and tension respectively.

Its application may be shown in the following case. Referring to Fig. 49, in which it is assumed that the wall is subject to combined earth and water, the diagrams have been worked out in the same way as Fig 46, the stresses being measured according to the scale.

The water pressure is considered as acting one-third the distance up the face of the wall, and perpendicular to that face and equal in intensity to

$$\tfrac{1}{2}w.h.l=\frac{62{\cdot}5\times10\times1}{2}=3125 \text{ lbs.}=28{\cdot}4 \text{ cwts.}$$

In measurement it will be found that when empty, viz. with earth pressure only, the resultant cuts the base at 1·3 ft. from the centre, and when full 1·35.

Using equation $\frac{W}{A} \pm \frac{M}{Z}$ in the case of there being no water pressure we have—

$$\frac{30{\cdot}4}{3{\cdot}5} \pm \frac{30{\cdot}4 \times 1{\cdot}3}{\frac{1}{6}(1 \times 3{\cdot}5^2)} = 8{\cdot}7 \pm 19{\cdot}36 = 28{\cdot}06 \text{ cwts.}$$

or 1·4 tons per square foot compression at toe and 10·60 cwts.

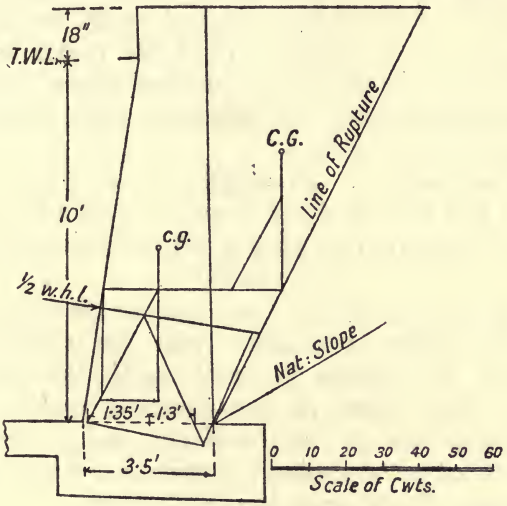


Fig. 49.

tension at inner edge per square foot. (The table in previous chapter in reference to tensile strength of mortar in the case of masonry walls should be referred to.) If the water is up to the level shown,

$$\frac{W}{A} \pm \frac{M}{Z} = \frac{36}{3{\cdot}5} \pm \frac{36 \times 1{\cdot}35}{\frac{1}{6}(1 \times 3{\cdot}5^2)} = 10{\cdot}03 \pm 23{\cdot}8 = 33{\cdot}83 \text{ cwts.}$$

or 1·69 tons compression and 13·77 cwts.=·69 tons tension per square foot.

Profile.

Walls may be curved on face. This construction is used in special cases, as will be pointed out, although the expense is more than a wall with a straight face in most cases. The

reader is referred to Fig. 50. A wall of suitable thickness and rectangular cross-section is set out by means of Equation 1, and its centre of resistance found at F. Q N should be made three times E F and E Q should be joined. We have an irregular prism, and that is the part which may safely be removed, because the centre of gravity of the prism is directly over F.

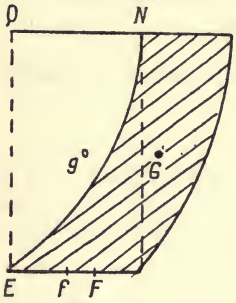


Fig. 50.

Now, to find the centre of gravity of such curved walls, we proceed as follows. Find the centre of gravity of the complementary rectangular wall=G, then to find the distance $g-G$, there is an equation, viz :—

For curved walls $$gG=\tfrac{3}{8}QN \quad . \quad . \quad . \quad . \quad (9)$$

The curve E N may be either circular or parabolic in Fig. 49.

Now the moment of stability of any vertical wall is equal to

$$S=gwhbt^2 \quad . \quad . \quad . \quad (10)$$

($g=\cdot 375$ in English engineering practice).

Obviously, then, we must fulfil the condition that S=moment of stability is equal to M, the overturning moment. The whole of our previous and subsequent reasoning depends on this condition, and to ensure that such is fulfilled is the engineer's business. When designing, both M and S should occur in pounds=feet.

Failure.

Let us consider how a wall may fail. It may (1) shear, (2) topple over, (3) crush the outer joints and tear the inner ones, (4) slide on the surface, (5) slide on the bed joints. In the case of (2) our graphical construction will avoid this, if properly done, by ensuring that the resultant shall fall in the middle third of the wall. In the case of (3) the Equation 4 decides this. In the case of (1), when a wall is homogeneous, or without joints, such as is the case with monolithic concrete, sliding will occur by one minute particle sliding upon the next, which is known as shear. Cement mortar is a good precaution against any liability to slide by confining this failure to shear,

which, as shown in the following table, is an unlikely occurrence.

Safe strength of cement concrete in tension and shear.

TABLE V

Mixture.	Tensile strength at 3 months. lbs. per sq. in.	Tons per sq. ft.	Shear. Tons per sq. ft.
1–4 1–6 1–8	60	3·8	3·5 2·5 1·75

Sliding.—When walls are subject to lateral pressure, sliding, at a joint in the wall, will depend upon the weight of the wall, the area of the joint, and the material of the surfaces in contact, and is to be overcome by the outside force P. This might be written :—

$$W \times F = P$$

$$\text{therefore } F = \frac{P}{W}$$

F is a factor representing the friction of the surfaces in contact, and is called the coefficient of friction.

Its values are given in the table below.

For common brickwork in mortar it is generally taken as 0·5. As an example say :—

$$W = 1680 \text{ lbs.}$$

$$P = (19{\cdot}5 \times 5) + (15{\cdot}6 \times 5)$$

$$= 175{\cdot}5 \text{ lbs.}$$

$$\text{and } \frac{P}{W} = \frac{175{\cdot}5}{1680}$$

$$= 0{\cdot}105$$

which is lower than 0·5, so that the wall is quite safe against sliding. In practice this figure is not allowed to exceed four-fifths of the coefficient of friction. This value is not approached until the angle which R makes with the horizon is less than 30°, and it is only necessary to make calculations for sliding when the inclination of R is less than 30°.

It should be noted, however, that this does not hold good at the point where the foundation rests upon the earth, as the coefficient is much smaller.

TABLE VI

COEFFICIENTS OF FRICTION (MORIN)

	F
Masonry and brickwork (dry)	0·6 to 0·7
,, ,, with wet mortar . . .	0·47
,, ,, with slightly damp mortar .	0·74
,, ,, on dry clay. . . .	0·51
,, ,, on moist clay . . .	0·33

Buttresses.

We have previously considered walls of regular rectangular section in plan. This is not always the case, and a series of projections are carried out from the wall at regular intervals, and they are known as *counterforts*. Similar projections on the face of the wall are called *buttresses*.

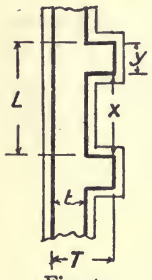


Fig. 51.

When walls are built with large counterforts there is a slight saving of material in the end. Assume a wall with such counterforts placed x feet apart.

Let t=thickness of the wall between counterforts.

,, T= ,, ,, including counterforts.

,, y=width of the counterfort (see Fig. 51) and g a coefficient = ·375.

Then the moment of stability of a vertical wall between the counterforts is equal to

$$S_1 = gwhxt^2 \quad . \quad . \quad . \quad . \quad (11)$$

and at the counterforts its value is

$$S_2 = gwhyT^2 \quad . \quad . \quad . \quad . \quad (11)$$

Now add these two moments together and divide their sum by the total length of wall=L. Then we have a formula giving the *mean* stability per *unit* length, and its value is

$$gwh\frac{xt^2+yT^2}{x+y} \quad . \quad . \quad .$$

By transposing we have

$$t_1 = \sqrt{\left(\frac{xt^2+yT^2}{x+y}\right)} \quad . \quad . \quad . \qquad (12)$$

and this may be compared with our previous reasoning to see what saving of material we have. Professor Rankine terms this the *equivalent uniform wall.*

The ratio of masonry in a counterforted wall is to the masonry in a plane wall

$$\text{as } (xt+y\mathrm{T}) \text{ is to } (x+y)t_1 \quad . \quad . \quad (13)$$

General Construction—(Practical)

In the case of retaining-walls for railways and roads and similar cases, where water is not met with, the work does not present any special difficulty. They are usually battered on the face, to save material and also to give a more pleasing appearance to the work as a whole. The bed joints of the face work should be perpendicular to the face, and in the case of curved walls this must be the rule also. Batter walls are set out with a batter rule, which is simply a plumb rule with one side cut to the required batter. In the case of curved walls (curved in cross-section on face) a face mould has to be employed. This is a templet set to the correct curve which, for the sake of simplicity, should be circular, as nothing is gained by the parabolic form in this connexion. A templet would be about 10 ft. long, and assume the radius of the face to be 140 ft.$=r$, l=length of templet=10 ft., x=rise or versine of the curve.

$$x=r-\sqrt{r^2-\left(\frac{l}{2}\right)^2}=140-\sqrt{19600-25}=\cdot0893=1\tfrac{1}{16}''$$

The value of $m=\sqrt{\left(\frac{l}{2}\right)^2+x^2}=\sqrt{25+\cdot0893^2}=5\cdot0008$

and y^2 versine of the chord of half the arc$=r-\sqrt{r^2-\left(\frac{m}{2}\right)^2}$

$$=140-\sqrt{140^2-2\cdot5004^2}=\cdot02^1=\tfrac{1}{4}''.$$

Having x, l, and two values of y, there are five points from which the templet may be set out. Works circular on plan, if very large, are set out with a theodolite in the customary manner, but in the case of short radii up to

60 ft. the instrument shown in Fig. 52 is used, on the system set out in Fig. 53, and described as follows by a correspondent in *The Building World* :—

" To set out a curve in a road, footpath, or kerb gracefully is no easy task. The accompanying drawings illustrate a simple instrument by which a curve of any radius from 10 ft. to 60 ft. can be set out which will not have the irregular effect too often seen. It is based on the platelayer's method of setting out a curve without a theodolite, by off-sets from equal chords.

" The instrument is constructed similar to a tee-square, and shown complete in Fig. 51 : A is an oak lath, $\frac{5}{8}$ in. thick, 3 ins. wide, and 7 ft. 4 ins. long ; at one end is fitted the cross-piece B, of oak, $\frac{5}{8}$ in. thick, 3 ins. wide, and 3 ft. long. The two pieces are halved and fixed together with screws,

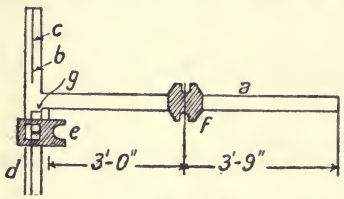


Fig. 52.

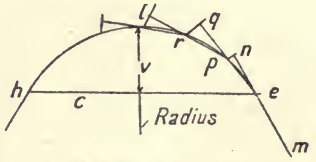


Fig. 53.

and must be perfectly square. Over the arm B is placed the slider D, with pointer and notch for an ordinary $\frac{1}{4}$-in. diameter chain arrow. The slider is a band of sheet brass, $\frac{1}{16}$ in. thick, $1\frac{1}{2}$ ins. wide, and bent to slide over the arm, and should be a good fit. On the face of the band is soldered a strip of sheet brass 3 ins. wide and $\frac{1}{8}$ in. thick, the edge on the inside projecting $\frac{1}{2}$ in. beyond the edge of the arm. A square opening, $\frac{1}{2}$ in. wide, is cut in the centre of the brass plate, exposing the scale, and at the middle of this opening is soldered a pointer. The notch E for the chain arrow is cut with a $\frac{1}{4}$-in. diameter round file, the centre of the notch must be perfectly in line with the pointer, and cut to a depth to allow the arrow to bear against the edge of the cross-arm B. On the outer edge of the slider is soldered a strip of brass $\frac{3}{4}$ in. wide, 1 in. long, and $\frac{1}{4}$ in. thick, and at the centre of this strip is drilled and tapped a hole for a

¼-in. clamping screw. This screw is to fix the slider when set for laying out a curve of given radius. On the face of the cross-arm is a scale C, divided into inches and decimals of an inch. The scale is either painted direct on the arm, or on paper pasted on and varnished. The scale is adjusted so that when the slider is close against the long arm, the chain arrow against the long and cross-arm in the angle G, the zero of the scale is immediately under the pointer. A brass plate F, ⅛ in. thick, 1½ ins. wide, and 3½ ins. long, is fixed with screws on the long arm; the centre line of the plate must be exactly 3 ft., and ¼ in. from the inner edge of the cross-arm. On the centre line of the brass plate a slot, ¼ in. wide, is cut with a ¼-in. round file. The slot is cut to the edge of the long arm, to allow a chain arrow to rest against the edge of the arm. Great care must be taken that the slots are cut to the dimensions, for on this the accuracy of the instrument depends.

"Before the instrument can be set to lay out a curve, the radius of the curve to be laid out will have to be found by the rule: $\frac{C^2}{v}+v$, where C equals the half of the chord and v the versed sine, or the height of the curve, at the centre of the chord.

"Fig. 53 will explain the method of finding this data in actual work, H E being an assumed curve to be laid out. Stretch a line from E to H, the commencement of the two ends of the curve; measure this length, the half of which will be C in the rule. From the centre of H E, measure to the point L, the highest point of the curve; this height will be v, or versed sine in the rule.

"Assuming H E to be 40 ft. long, and the length V L 10 ft., the radius of the curve will be $\frac{20^2}{10}+10=50$ ft. The radius of the curve being known, the half-length of the off-set from the tangent will be necessary, which is found by the rule $R-\sqrt{R^2-L^2}$, where R is the radius of the curve, and L the length of the tangent, or the length of the arm of the instrument from F to G in Fig. 52. The half-length of the off-set for a radius of 50 ft. by this rule will be:

$600-\sqrt{600^2-36^2}=1{\cdot}08$ in. In the working of this rule, all dimensions are reduced either to feet or inches; in the example they are reduced to inches.

"The following table of off-set lengths is for a 3-ft. tangent, for radiuses from 10 ft. to 50 ft., calculated by this rule; from which an extended chart, giving intermediate radii, can be set out on squared paper, thus avoiding calculating off-set lengths each time a curve is set out:—

TABLE VII

Radius. ft.	Half-length of Off-set. in.	Full-length. in.
10	5·53	11·06
15	3·64	7·28
20	2·44	4·88
25	2·17	4·34
30	1·8	3·60
35	1·55	3·10
40	1·35	2·7
45	1·20	2·4
50	1·08	2·16

The most convenient chart would be the half-length off-set, and for the full length it is simpler to double the values in the chart.

"The length of the first off-set, P N in Fig. 53, where the curve starts from the straight line, is the length found by the rule, and is half the length of the succeeding off-set R Q.

"To set out a curve, an arrow is driven in at the start of the curve at E, and another at M, 3 ft. back on the straight line. The slider on the cross-arm is set with the pointer over the half-length of the off-set, corresponding to the radius of the curve."

Construction in General

The provision of adequate foundations is very important to the stability of any retaining-wall. They must be stepped out in front and are sometimes similarly treated at the back.

It is a mistake to economize material in this direction, because lack of weight at the base of the wall, at the expense of reducing the bottom width, raises the centre

of gravity higher than it otherwise would be, and the tendency to move outward at the top will be the result In some cases the top is widened, usually to provide a substantial coping, as in the case of dock walls, where they have to carry fairleads, mooring-rings, etc., and the base reduced to save expense. Generally it is better to make the top as narrow as possible and spread out the base to distribute the weight over a greater area. Hollow walls may in some cases be constructed in order to secure a greater supporting area without increasing the quantity of material.

Theoretically a triangular section is quite safe, but for obvious reasons cannot be adopted in practice, but in the case of walls which retain more or less dry earth there is no reason to depart greatly from it.

Spreading out the foundation courses brings the centre of resistance nearer the middle of the base than it is in the body of the wall, and the pressure is distributed over a greater area than that of any bed joint in the body of the wall.

Drainage.

Adequate drainage is essential to the durability of retaining-walls. Along the front of the base there should be a continuous drain. This may be of stones filled into a cutting, or open-jointed fireclay pipes packed round outside with stone and led to a suitable outfall. The water behind the wall must be dealt with too, because, if allowed to collect, not only does the water, exerting as it does its full pressure horizontally, put an undue load on the wall, but it also gets into the joints and may cause damage in frosty weather. In cases of very wet ground it is much better to place a layer of dry stone filling at the back of the wall, with a box drain at the foot, but in most cases weep-holes suffice. These simply consist of rectangular openings in the wall from front to rear, in depth equal to that of the course and in width about 2 ins. They should be distributed at regular distances and in number equal to one for every four square yards of wall face. In any case the backing should be carefully rammed in layers, in such a way that each layer slopes away from the

wall downwards, at about 1 in 5. In the case of surcharged walls a catch-water drain must in all cases be cut along the slope to intercept any surface water, any springs being led into it. Walls which are very heavily surcharged and would at any time be likely to have an undue thrust, due to the backing being of an unstable nature, must be either designed as water walls or must be tied back by what are called land ties. They are also used in sea defence works. The tie consists of a rod of iron attached to the wall by a nut and large X-shaped washer, and similarly attached to a large iron plate, placed some distance back in firm earth or concrete, and have a slope at a small angle from the vertical. The holding power of such ties may be found from the equation—

$$H = W \cdot \frac{x_2^2 - x_1^2}{2} \cdot \frac{4 \sin \Theta}{\text{Cos}^2 \Theta} \quad . \quad . \quad (14)$$

where H=holding power per foot of breadth of plate, W=weight of earth per foot cube, and Θ its angle of repose, while x_1 and x_2 are the depths of the upper and lower edges of the plate below the surface. Again, if y=depth of the centre of pressure of the plate below the surface of the ground, its value is

$$\cdot 666 \frac{x_2^3 - x_1^3}{x_2^2 - x_1^2} \quad . \quad . \quad . \quad (15)$$

and this point y is the centre of the hole through which the tie-rod must pass. They are generally placed through the wall at a height equal to two-thirds the height of the wall above the foundation, but if the wall depends *solely* on the ties, one-third the height.

Inverts.

Sliding of walls in slipping ground is prevented by *undersetting*, a process consisting of horizontal buttresses.

They have been used in railway cuttings, and sometimes extend the whole way under the rails to the opposite wall. This construction was applied in the Blisworth cutting of the London and North Western Railway. The base of a retaining-wall may also be prevented from sliding forward by a series of horizontal struts of masonry which abut

against rectangular masses, whose resistance to displacement will depend on the same principles as the holding power of anchor plates. In the same way in cuttings having two retaining-walls, cast iron or lattice-steel ribs may be used for the same purpose, but have not often been employed owing to expense. Such ribs would abut against the wall at a point two-thirds the height in distance from the base.

Stability.—The principle of considering the angle of repose which the material composing the backing of a retaining-wall will take has been discussed, assuming that the angle of rupture is half this angle. A knowledge of the actual pressure of earthwork is of the greatest importance, and in no branch of engineering is there such a scarcity of EXACT data as that which refers to the actual lateral pressure to which retaining-walls are subject. The case has usually to be taken on its own merits, and although tables have been given dealing with the angle of repose, yet the wise engineer will carefully ascertain this by trial heaps of the ACTUAL material which will ultimately form the backing of the wall. Again, a material perfectly suitable for backing up a wall under favourable conditions may become saturated before a sufficient time has elapsed for it to become thoroughly consolidated, and in consequence reduced to such a condition that the thrust on the wall approaches the intensity of that due to water. Clay, especially when wet, is most treacherous and will slip freely; moreover, it is liable to expand on exposure. On the other hand, it has been well proved that* if retaining-walls are designed on the assumptions previously set forth the result will be to err largely on the side of safety, and the bold engineer could greatly reduce the section so found. But it is not wise to go so far as this, because there are other points, such as vibration and inequality of foundations, that have to be contended with, and their effect is little known, and it is more prudent for the engineer to increase the first cost of works of this kind rather than

* "The Actual Lateral Pressure of Earthwork," by Sir Benjamin Baker (*Trans. I.C.E.*, Vol. LVX).

court failure and disaster. Another way of considering the design is to assume a fluid pressure, but in place of the usual 62½ lbs. to consider it as 20 lbs. It is a useful check on the other calculations. It is quite safe to design a wall which in theory would be just about to overturn. A cause of failure, however, must not be ignored, and that is of filling in before the masonry is solid, and to ensure that the mortar shall set, and set quickly, only Portland cement mortar should be used.

CHAPTER IV

DOCK, QUAY, RIVER, CANAL, AND SEA WALLS

IN the previous chapter we have considered the design of walls in general and also walls which may be termed "dry" walls, viz. not in contact with water. Now, such walls as are denoted above are in contact more or less with water, and although their design is nothing very much out of the ordinary when the circumstances of the case are considered, yet the carrying out of the works is work which demands the utmost engineering skill and cannot be adequately treated on paper, although various suggestions will be put forward here. Mr. Colson says :—

"Wharf and dock walls generally are subject to conditions altogether different from those affecting ordinary retaining-walls, and have therefore to be dealt with exceptionally. Heavy traffic, often setting up strong vibrations, the erection of buildings and depositing of heavy merchandise on the wharf, have to be taken into consideration when determining the section of the wall to be adopted. In addition there is a contingency of the dock having at some time or other to be emptied for cleaning or repairs, in which case additional strain would be thrown upon the walls, as they would then probably be called upon to bear an abnormal pressure, bearing in mind that it is practically impossible to prevent in all cases the accumulation of water at the back of the dock wall."*

Dock and quay walls have to be very massive, and it is frequently the case with docks that they are built on very bad ground, often the deposition of dredging operations, and in consequence the wall must be designed to withstand

* *Notes on Dock Construction* (Longmans).

water pressure from both sides.* Again, water is bound to find its way into the foundation of the walls, and the strata gets soft and the pressure is no longer uniform, and unequal settlement takes place, ~~an example of which took place at Southampton,~~ where the toe of the wall slipped gradually forward. Again, the front of a quay wall must be designed with regard to the convenience of the shipping which will use the dock. An excessive batter tends to keep the vessel too far from the wall at the top, causing rubbing. Curved walls of short radius are equally disadvantageous, but when a radius of two and a half to three times the height of the wall is given and the centre is plumb with the top of the cope stone a very good section results, except in the case of tidal basins, where a curve will throw the vessel right off the quay at low tides. In such cases flat batters of about 1 in 20 are admissible only. Another element of design lies in the fact that in the case of porous subsoil the water will rise at the back of the wall to a height equal, or nearly so, to that at the front, and the masonry, instead of having its full weight due to gravity, will have an effect equal to the same less $62\frac{1}{2}$ lbs. per foot cube, although the filling, if of rubble, will be correspondingly reduced and the thrust will remain unaffected. If the walls are founded on rock or clay there is no reason for the water to get under the wall, and if properly drained at the back by a rubble drain the water pressure will always act contrary to the thrust of the backing, so long as it remains at a constant level, which, however, as was pointed out, is not always the case. In the case of water on both sides of the wall, the same being built with buttresses, not carried up to the full height and having a flat top, the pressure of water on the top of such buttresses will increase the stability, in the same way as off-sets, as shown in a previous example. It will be shown subsequently that off-set walls are a very favourite form in dock and quay walls.

It is hardly within the province of this book to consider such cases as the depth of water required alongside any quay necessary for the vessels using those

* Viz. :—from the land side when the dock is empty.

quays; suffice to say that this depends on the range of tides and maximum load-draught of the largest vessels to be accommodated. We proceed, therefore, to consider the very important question of getting in the foundation of walls in water. This is perhaps the most important of all engineering works, and is work that demands great skill and experience, as before stated. Cofferdams have generally to be employed. A cofferdam essentially consists of two parallel rows of main piles and sheet piles, enclosing between them a vertical wall of clay puddle. The simplest

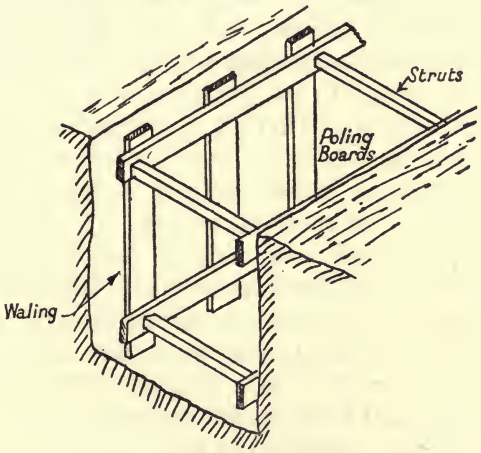


Fig. 54.

kind of foundation, of course, consists of a trench dug to the required depth and timbered. This is shown in Fig. 54, in which there will be walings, poling boards, and struts, as shown. The idea will be quite apparent to the reader, and can be elaborated on according to the different circumstances under which the work will be carried out. Small quantities of water may be pumped out, and there is not any special difficulty in the work. Where the water is permanently or will periodically rise above the level of the foundations of any wall, then the circumstances are altogether different. In making a cofferdam the main piles have to be driven first. The fundamental equation in

reference to foundations of all classes is that enumerated by Professor Rankine.

We have x=depth of foundation in feet.

w=weight of 1 cub. ft. of the earth (or rock).

ϕ=its natural angle of repose (p. 42).

Then if we have a load (say that of a building, bridge, pier, or engine foundation)=P_0 per square foot in lbs.,

we have $$\frac{wx}{P_0} \geqslant \left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 \quad . \quad . \quad . \quad . \quad (16)$$

which means to say that the sign $\geqslant$ signifies that the first term in the equation must be at least equal to the second or greater than it (never less); this equation is mostly used in reference to soft earths or those in which there is a liability to slip owing to the presence of water, the formula being hardly necessary with rock or compact earths, because the natural practical depth of the foundation is almost *sure* to be greater than x, being about 3 ft. for sand and 4 ft. for clay, in this country at all events. In America and in cold climates generally we may go further to avoid the effects of severe frost. In firm earth the usual safe intensity of pressure would be between 2500 and 3500 lbs. per square foot. All trenches for founds must be carefully drained before putting them in; and if in soft ground, they may be made deeper and filled up with more stable earth.

In this case if w =weight per foot cube of the filling,

w_1= ,, ,, earth,

ϕ =angle of repose of the former,

ϕ_1= ,, ,, latter,

then the values $\left(\frac{1-\sin\phi_1}{1+\sin\phi_1}\right)$=K and K_1 for each of the two materials are found, and the following equation results also giving the depth of the proposed foundation *before* the filling is put in. The value is—

$$x_1=\frac{P_1K_1^2}{w_1-wK^2} \quad . \quad . \quad . \quad (17)$$

P_1 being the greatest intensity of pressure expected. The filling must be well rammed.

Piles.—Piles are driven by a pile-driver; this may be a hand-

worked machine in the case of small works, and in the case of large ones steam-driven. The whole essentially consists of a timber or steel frame, having a pulley at the top and a steam or hand-worked winch at the base. From this and over the top pulley is worked a large weight, up and down vertically on guides which form part of the machine. The pile is placed vertically under this weight, called the ram, and it is raised and suddenly released, with the result that it falls on the head of the pile and drives it. There are also some pile-drivers which act directly on the pile by steam, like a steam hammer. A pile-driver of ordinary construction is shown in Fig. 55.

For application to such an engine we have the following formulæ—

W=weight of ram (lbs.).
H=its fall (in ft.).
w=weight of pile (lbs.).
P=length of pile (ft.).
A=sectional area of pile (sq. ft.).
E=modulus of electricity of the pile.
D=set of pile by the last blow (ft.).
L=greatest load on the pile (crushing).
l= ,, ,, (safe).*

Then $$L=\sqrt{\left(\frac{4EAWH}{P}\right)+\left(\frac{4E^2A^2D^2}{P^2}\right)}-\left(\frac{2AED}{P}\right) \quad . \quad (18)$$

Again, if A=sect. area of pile in square inches,

$$W=w\left(\frac{wH}{5AP}-1\right) \quad . \quad . \quad . \quad (19)$$

A short fall with a heavy ram has more effect than a long fall and a light one, the number of ft.-lbs. of energy in all cases being the same.

In the *Transactions of the Institution of Civil Engineers*† we have the following conclusions, viz. :

1. More waste of labour is caused by blows than by constant, steady pressure.

2. One blow exercising the same mechanical power as

* Usually=$\frac{1}{6}$ L or $\frac{1}{3}$ L in specially good places.
† Vol. XLIV.

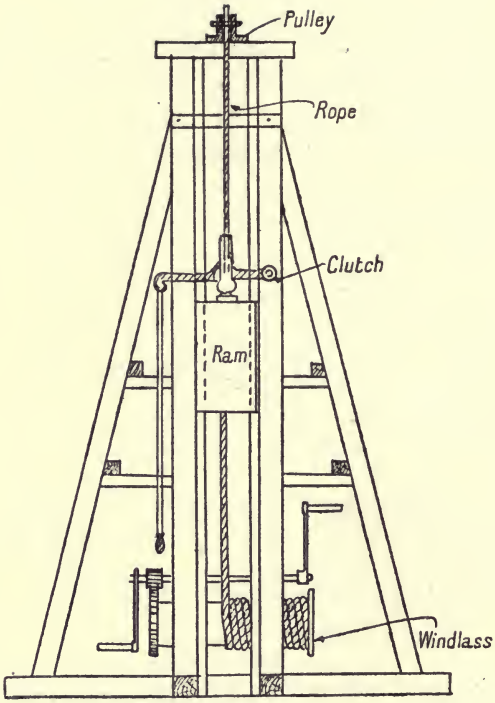


Fig. 55.

that of a steady known pressure will not produce the same effect as that pressure.

Elm or pitch pine are generally used for piles. It should

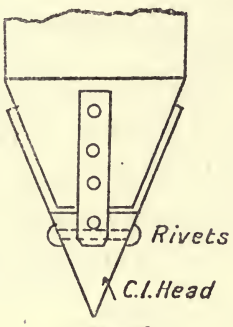


Fig. 56.

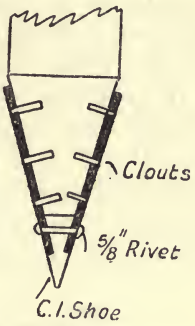


Fig. 57.

be straight grained and have all the bark stripped off. The piles must be prepared as shown in Figs. 56, 57, 58, 59 with shoes and a ring at the top to prevent splitting

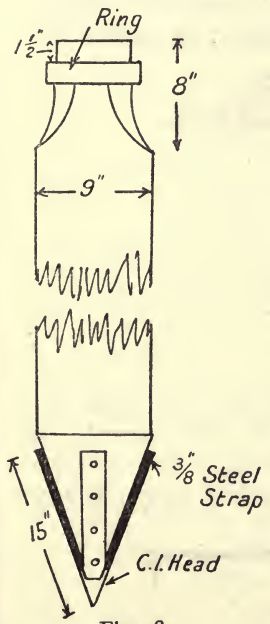


Fig. 58.

Greenheart has been used for piles; it is very hard and durable, but expensive. In large works the piles may be withdrawn and reused in other places. This, when repeated often, means damage to the piles, and the attention of the reader is drawn to the modern method of steel piling. This consists of using steel beams and various sections put up for the purpose, as shown in Figs. 60, 61, 62.

Steel Piling.—The "Universal" joist steel sheet piling is composed of 15-in. by 5-in. British standard joists and a standard interlocking member made from a $6\frac{3}{16}$-in. by 4-in. joist turned over by a special process. The interlocking member or clutch is run on to the flange of the joist before driving, sc

that each pile consists of one joist and one clutch. This form of interlock is suitable for all British standard joists, so that any desired width of joist can be used so as to make up an exact distance.

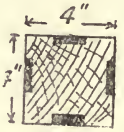


Fig. 59.

The method of driving is to commence with a plain joist, great care being taken to see that it is perfectly plumb all ways, then continue with one clutch and joist together, driving the clutch over the flange of the joist last driven; this prevents the interlock filling with soil. A steel driving cap is used to protect the heads of the piles where it is desired to use the piling repeatedly, but in some cases for

Fig. 60.

permanent work, where the driving is very free, this may be dispensed with. Owing to the great accuracy with which the piling is made, the clearance is sufficient to enable the

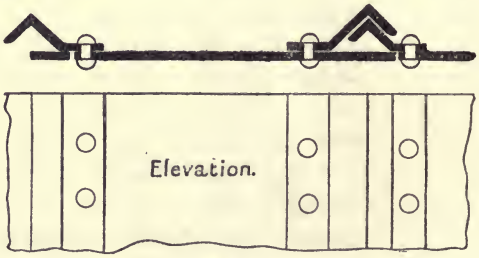


Fig. 61.

piles to drive freely, and yet small enough to prevent the water passing through. Each clutch has four points of contact, so that it is very difficult for the water to pass through, owing to its having to take such a long journey

Fig. 62.

round the clutch ; but should there be a small space, it gets filled up in the course of a few hours from the dirt in the water, or, where there is no dirt, from the road sweepings which may be thrown in.

For cofferdam work it is usual to set up the piling in position and work round it with the piling plant, so as to get all the piles down together and ensure closing. This piling may be used for all classes of cofferdams, retaining-walls, mine shafts, sea defence works—in fact, in all cases where it is necessary to retain earth or water, or the two combined. It has great strength, and consequently can be driven to great depths.

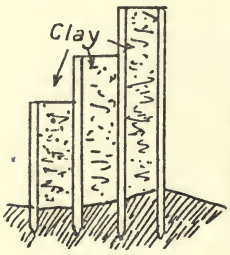


Fig. 63.

Running sand, which is always a difficult problem with timber piling, is a simple proposition with Universal joist piling, as the displacement is so small in comparison that it can be easily and rapidly driven, where a timber pile refuses. The same applies to all compact soils.

Cofferdam.—Now as to the actual cofferdam. The upper walings of the two rows of piles are tied together by cross-beams, which support a plank staging for the use of the workmen. The main piles in one row are usually 4 or 5 ft. apart. The ground is excavated between the rows of sheet piles till a sufficiently firm bottom is reached and the puddle rammed in layers. Cofferdams are usually made of a thickness equal to the height above ground, when this does not exceed 10 ft., and if greater,* 10 ft. $+\frac{1}{3}$ the height above 10 ft.† With high dams over 15 ft. or so, three or more parallel rows of sheet piles are driven to divide the thickness of the dam into a number of sections, the outermost being to the full height and the others lower in succession, as shown

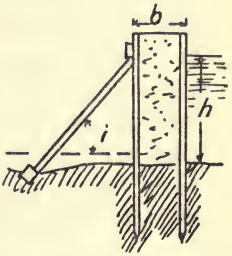


Fig. 64.

* And not strutted.
† Rankine, *Civil Engineering*, p. 612.

in Fig. 63, because a thickness of 5 ft. of puddle will ensure water-tightness and the rest of the dam is mainly for stability. A cofferdam may, however, be strutted against such lateral pressure and in consequence reduced in section, the idea being shown in Fig. 64. Then if $w = 62\frac{1}{2}$ lbs.

Total water pressure on dam P

$$= \frac{wbh^2}{2} \quad . \quad . \quad . \quad . \quad (20)$$

Moment due to water pressure M

$$= \frac{wbh^3}{6} \quad . \quad . \quad . \quad . \quad (21)$$

and thrust along the strut

$$= \frac{\text{M} \sec i}{h_1} \quad . \quad . \quad . \quad (22)$$

Dams form a very important adjunct in marine masonry construction for the purpose of enclosing parts where it is necessary to dry the bottom for the purpose of getting in foundations. They are temporary works, and, as such, are liable to be hastily and cheaply put up, unless the contractor is carefully watched, and failure of such works often means a great deal of damage and often loss of life. It is the worst possible engineering policy to economize in this direction, as it may mean greatly increased cost in the end, and the engineer is wise to bestow just as much (if not more) forethought on such preliminary works as the rest of it. Sheet piles are best made tongued and grooved by spiking 3-in. fillets to the sides of the pile. They will serve to keep the piles together under ground. Leakages* in dams are very

* The following instance of stopping a cofferdam leak by Col. O. M. Por, U.S. Engineers, taken from *American Architect*, August 1st, 1891, p. 69, is interesting:—

"The cofferdam referred to on the Sault Ste. Marie Canal consists of a clay wall having a minimum thickness of 8 ft. This clay wall is supported on each side by a line of cribwork filled with stone, and therefore pervious to water. A portion of the dam extends from the west end of the north wall of the present lock up-stream, along the middle of the canal for about 580 ft., and then turns squarely into the north bank of the canal. The leak occurred about 100 ft. east of where this turn was made, and was situated entirely beneath the cribwork, the bottom of which, at the site of the leak, was 22½ ft. below the surface of the water of the canal. The cribwork was not disturbed. At the deepest point the leak was at least 4 ft. below the

troublesome and likely to occur, especially through the bolt-holes which pass the bolts to tie the two sides together. Through bolts are best dispensed with where possible by the use of buttress piles driven outside, but this is not

bottom of the crib, therefore 26½ ft. below the level of the water in the canal, or about 9 ft. lower than the level of the water in the river below the lock.

"The rush of water underneath the dam was so great that bales of hay and gunny-sack filled with clay thrown into the cavity were carried through. By driving sheet-piling on the down-stream side of the space from which the clay wall had been carried away a sufficient obstacle was established to prevent hay in bales and clay in sacks from being washed through, and after some two thousand sacks of clay had been thrown in it became possible to fill the gap with loose clay. But the leak still remained too large for the pumps to handle, and to further reduce it, as well as to get something into place more substantial than the sheet-piling which had been driven at first, timbers 12 ins. square, sharpened to a chisel-edge at the lower end, and shod with iron, were driven with a 1900-lbs. ram between the sheet-piling and the cribwork, until they could be driven no further, when it was assumed that they were in contact with the rock formation, and a note made of the depth to which they had gone. The clay wall was then loaded with heavy masses of stone to compress it, and the result was the reduction of the leak to such a degree as to bring it easily within the control of the pumps, and nineteen days after the break the lock-pit was again empty of water. The volume of water in the pit was about 45,000,000 gallons, and this, in addition to the leakage, was handled by the pumps in less than seven days.

"Although the pumping capacity available was sufficient to easily keep the lock-pit empty, it was still very desirable that the leak should be entirely stopped, and it was believed that this could be done very soon after its exact location was ascertained. To find the crevice in the rock through which the water was passing, an iron rod was driven down through the clay wall in front of and close to the timbers referred to until it would go no further. The depth to which it penetrated was compared with the depth to which the timber had been driven, and the proper notes were made concerning any effect upon the muddiness of the leak. The rod was then withdrawn, moved laterally an average distance of 8 ins., and again driven to the rock. This operation was continued until at one place the rod went 2½ ft. deeper than at the distance of 8 ins. on either side of it. Here, then, was the crevice sought, and the next thing to be done was to fill it with clay or other water-tight material. It was nearly 30 ft. below the top of the clay wall, and more than 26 ft. below the level of the water in the canal. It was not practicable to excavate down to it, and no other mode known than that as 'stock ramming' seemed available by which to put the clay where it would do the most good.

"'Stock ramming' consists in simply driving a tube of requisite diameter to such a depth that its lower end will be at the point where it is desired to deposit the clay or other impervious material, and then forcing such material there in such a quantity as may be deemed necessary. In this case, the tube used was an iron pipe. An iron rod is used as a piston, and this piston is driven down as a pile-driver, the clay with which the pipe has been filled being forced out at the lower end. When the piston has reached the lower end of the pipe it is withdrawn and the pipe is again charged with clay and the operation repeated. This is continued till the clay rammed in forces upwards the whole column of clay above it, when, as is obvious, the operation can be carried no further."

always possible where the ground is bad and will not support the vertical and lateral thrust of the clay, and through bolts near the ground-line is the only available method. The use of iron plates 15 ins. square with holes cut near the upper edge and slipped on to the bolt at intervals are useful in localizing cavities set up by the sinking of the clay and in preventing leaks in consequence. Bolts must never be put in subsequent to the puddle, so that an excess of bolts should be originally provided for.

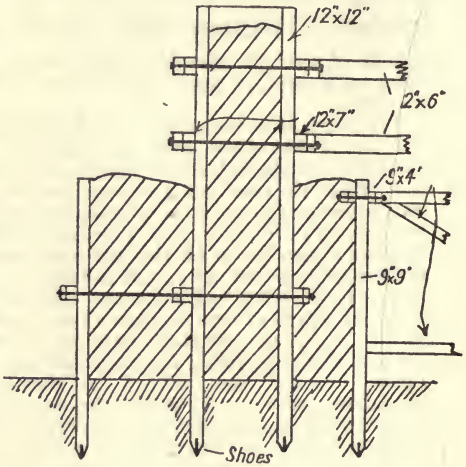


Fig. 65.

Machinery such as pumps should be kept clear of the dams whenever possible, due to the likelihood of damage by vibration. Regarding the forms of cofferdams, the engineer will find that

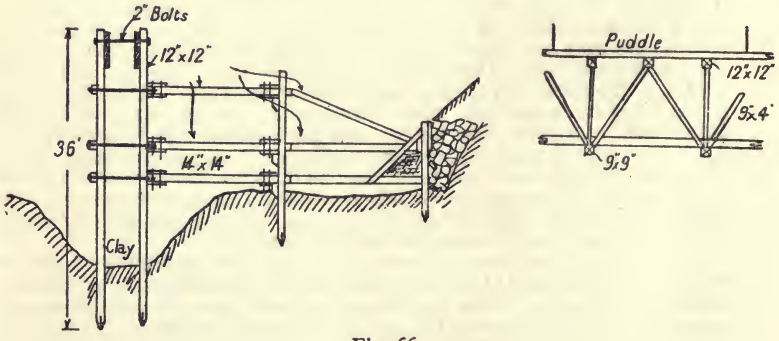


Fig. 66.

in practice no hard and fast rules can be laid down. An example is shown in Fig. 65 of a dam suitably strutted to some firm foundation at the back, such as

would be the case in a quay or river wall. In Fig. 66 is shown a dam of small section, but very rigidly strutted where water at the back would not collect to any great extent and the rise and fall of the head water small. Where the dam cannot be strutted to the back, as shown, the form of dam in Fig. 67 must be substituted.

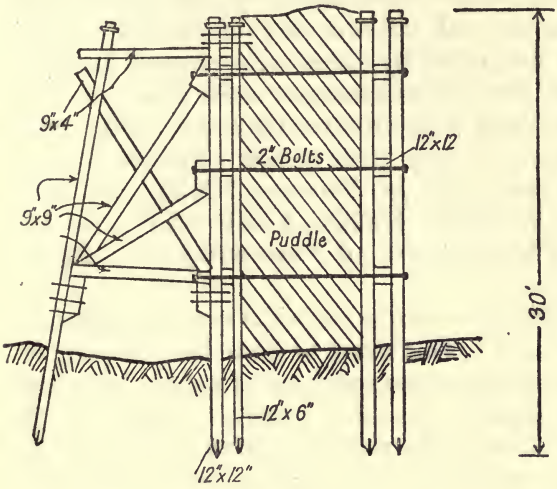


Fig. 67.

Single rows of piles may be used in the construction of dams and are often the only method of procedure where space for working is strictly limited, such as is the case with a navigable river, and also where the rise and fall of the head water is small, which makes it difficult to clear out mud and silt from between the rows of piles. The one thing essential is to get the piles closer jointed. Less timber is of course required and leaks, if any, are more easily dealt with. The piles must be driven very deep to prevent displacement by flotation. Tongued and grooved piles with iron tongues are generally used. Mr. B. B. Stoney in bridge work at Dublin used piles in sets of three dowelled together* by elm dowels sunk 3 ins. into each pile at intervals of 8 ft. The usual section of a single timber dam is given in Fig. 68, showing the use of buttress piles

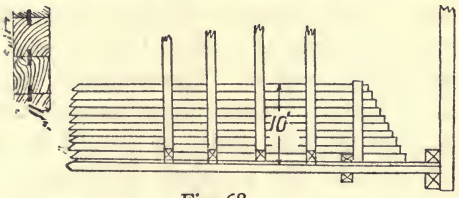


Fig. 68.

* *Trans. I.C.E.*, Vol. X.

about 10 ft. deep. It is not necessary to dwell on the subject of cofferdams, as an endless variety of instances could be quoted. The *Transactions of the Institute of Civil Engineers* are full of accounts of similar works, and should be consulted by those who require information. Moreover, the circumstances of each case are fully discussed there, and comparisons may be made with the circumstances of any new works. Information on the subject is very full in Volumes XXXI, LI, LIV, LVII; XXII–LV, XCII; XXXII, LXII, LXIV, and the past issues of *The Engineer* and *Engineering* may also be freely consulted.

Pumping.—When the dam is made the next business is to get rid of the water. This must be done by pumping, for which purpose pulsometer pumps slung on a chain have been largely used. Modern practice, however, tends to the employment of electrically driven sinking pumps of the centrifugal type, which may be made in such a way that the motor itself may be submerged. The Dublin Port and Docks Boards Engineer, Sir J. P. Griffith, during the present improvements, has a complete electric plant, consisting of portable locomotive boilers, steel chimneys, Parsons' turbines, and DC dynamos. The whole plant is shifted from point to point. In pumping plant for keeping foundations dry everything must be in duplicate, to guard against any breakdown, as the progress of the work will depend wholly upon the keeping out of the water, which, if it were to rise to a dangerous level, would probably ruin it. The engineer must always be ready to stop leaks. These unfortunately often occur in the form of springs and boils of sand-laden water, which may do damage to the pumps. It is quite impossible to give any definite directions as to the method of dealing with such contingencies, so very different will be the cause of the leaks; moreover, it is really out of the province of this book. The *Transactions of the Institute of Civil Engineers* may be again consulted with advantage, and also *Notes on Dock Construction*, by C. Colson (Longmans), and *Docks*, by Brysson Cunningham (Griffin).

Dock Walls.—We proceed to investigate some typical sections. In Fig. 69 is shown the masonry dock wall at the Alexandra Dock, Hull. The facing is ashlar stone* and the backing rubble chalk. Up to 14 ft. below the coping the upper portion projects 6 ins. beyond the lower part of the wall (a very usual form for quay walls), with a rubble backing and granite coping. Lias lime mortar was used on the works. The weepholes consist of iron pipes, and they are placed at 75-ft. centres, horizontally, and there is a rubble drain at the back of the wall 2 ft. 6 ins. square. Chalk backing, such as used in this wall, is durable enough when not exposed to frost.

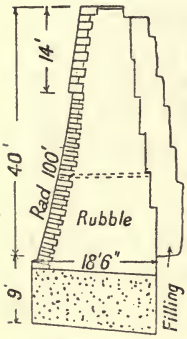


Fig. 69.

The Cork City deep-water quays are examples of masonry construction in part only, but are interesting. It may here be stated that nowadays it is very usual to make huge blocks of concrete in the form of stones for building walls, etc. The subject of concrete† is a very wide one, and cannot very well receive full consideration here, but the use of blocks when once made is precisely the same as masonry construction, and it is interesting to see how such blocks are used with resulting economy and rapid progress of the works. A typical section of the Cork north quays is given in Fig. 70, and in Fig. 71; the form of the blocks used is illustrated.‡

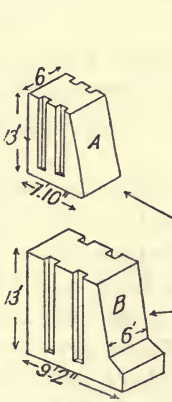


Fig. 70.

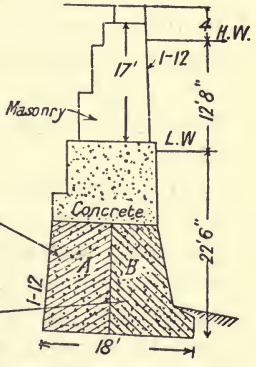


Fig. 71.

One-half the length of this quay was built to section,

* *Trans. I.C.E.*, Vol. XCII.
† Concrete blocks will be considered as an appendix.
‡ *Trans. I.C.E.*, Vol. C.

with two bottom blocks and one block on top. The average progress of the work was about 400 ft. per annum.

The block-wharf, a timber flooring resting on short bearing-piles, was 4 ft. 6 ins. above low-water of spring-tides, which level admitted of the blocks being made in advance of the rising tide. Each block was allowed twenty-eight days to set before being removed from the wharf.

The concrete for the blocks was prepared in one of Stoney's semi-cylindrical mixers with revolving blades, which delivered into small waggons running on a gantry over the block-wharf, and the concrete was tipped directly from these waggons into the block casings. The blocks when being removed were suspended by four stirrup-rods, in which rested two small wrought-iron girders running the full breadth of the block. These girders were placed in recesses constructed at or near the bottom of each block, and were withdrawn when the block was laid.

Vertical recesses, 10 ins. by 5 ins., were left in the sides of the blocks for the stirrup-rods, so that none of the suspending appliances projected outside them. When the blocks were laid in position, the recesses in each block corresponded with the recesses in the adjoining block, forming vertical spaces 10 ins. square, which admitted of dowelling the blocks together. They were filled with solid stone dowels, in lengths of 3 to 4 ft., for the entire height of the block.

The substructure for the entire length of 1,421 ft., with the exception of 30 ft. at the Water Street termination of the quay laid with concrete in position, consists of three concrete blocks each 6 ft. long lengthwise of the quay, weighing from 35 to 49 tons each. The blocks were made on a block-wharf on the opposite side of the river, and were transported and laid by a shears-float built for the work.

The concrete in the blocks was in the proportion of 7 to 1 by measure, namely, 5 parts of ballast as dredged from the river, 2 parts of limestone broken to pass through a 3-in. ring, and 1 part of Portland cement.

The foundation was gravel and sand, for the most part fine and compact. The superstructure was constructed of

dressed limestone, ashlar on face, laid in regular courses, and backed with 6 to 1 concrete.

In the same paper the cost of the blocks and facing is given as follows :—

	£ s. d.	£ s. d.
Blocks each—		
Concrete in blocks, on wharf, 7 to 1 (cement 48s. per ton), "1 ton of cement to 6 cub. yds. of block," 12 cub. yds., at 18s. .	10 16 0	
Shears-float, lifting and laying blocks, at £3 per block (including wages when idle), equal to 12 cub. yds., at 2s. 6d.	1 10 0	
Filling dowels and adjusting upper blocks, at 20s. per block	0 10 0	
		12 16 0
Superstructure—		
Ashlar facing (say 2 ft. deep), 15 sq. ft., at 6s. 3d.	4 13 9	
Concrete backing, 6 to 1, 3·21 cub. yds., at 18s.	2 17 9	
Granite coping, 10·76 cub. ft., at 4s.	2 3 5	
		9 14 11
Surface metalling (80 ft. by 9 ins.) 2·22 cub. yds, at 4s. 6d.		0 10 0
		£23 0 11

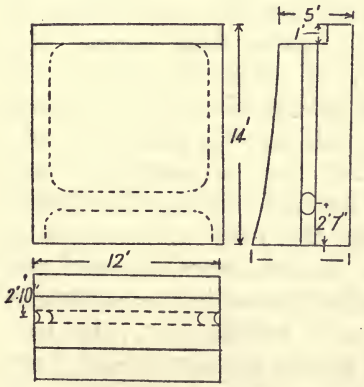


Fig. 72. Fig. 73.

Large concrete blocks of 70 tons each were substituted for stone in the New York River front wharf wall, their form being shown in Figs. 72 and 73. They were moulded in timber casings, and were composed of 1 cement, 2 sand, 5 of stone. Vertical grooves were moulded on each side and an opening

formed through the block near the bottom for housing the lifting-chains; when lowered into position a rope was made fast to one end of the chain and was afterwards withdrawn, its sole object being to allow the chain to descend only as fast as it could be withdrawn and so prevent any fouling. The blocks rested on pile-heads, with the intervention of 2 ins. of slow-setting cement in bags to form a cushion and ensure regular bedding. The chain-grooves were filled with concrete. Some of the Dublin Quay walls* were built in a similar way by Mr. Bindon Stoney; but they contained 350 tons of stuff, and were true masonry blocks. Their size was 27 ft. high, 21 ft. 4 ins. wide at the base, and 12 ft. 9 ins. at the top, by 12 ft. long on the face. The outer face of the blocks was formed of limestone set in 4 to 1 cement mortar, while the hearting was random rubble, the stones being just as they came from the quarry. The whole was made compact with small stones and a generous use of concrete, 7 to 1, the ballast being dredged on the site. They were allowed ten weeks to consolidate. The form is that shown in Fig. 74, the grooves being 3 ft. wide and 1 ft. 6 ins. deep, which were filled with concrete when the blocks were placed, so as to form a key. The manipulation of such blocks was, of course, a big job, and to do so cast-iron girders were built into the bottom of each block. To these were attached 5-in. wrought-iron bars 31 ft. long, with tee ends both top and bottom. The blocks had four rectangular holes 20 ins. by 11 ins., and each girder two holes, into which the tee sections were dropped and given a quarter turn, so that they locked in the girders for lifting. They were subsequently withdrawn. The bars were stressed during lifting to 8 tons per square inch, and,

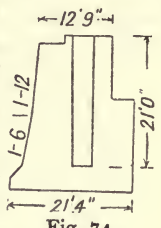


Fig. 74.

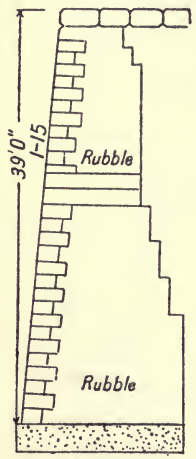


Fig. 75.

* *Trans. I.C.E.*, Vol. XXXVII.

being only iron, it appears a bold venture. The blocks were lifted and transported by a shear pontoon having two shear legs, one for lifting and the other for a counterbalance tank. In Fig. 75 is illustrated a typical example of a masonry quay wall as executed at Glasgow under quite ordinary conditions. In the Liverpool docks may be seen all sorts of quay walls, but one which is somewhat novel is the facing of the Herculaneum dock, which was excavated in soft sandstone.* It is shown in Fig. 76. The wall is thin and is dowelled into the sandstone by dovetail dowels, as shown, placed at 20-ft. centres. Blocks of ashlar cut to this size and shape were used. The process is called *veneering*. Of course, there are very many sections of walls which will present themselves to the reader, and it is useless to dwell on a subject whose range is almost infinite. It may be well to point out, however, a form of wall now much in favour for docks and quays, the theory of which has already been discussed, and consisting of arches and piers.† A saving of material results from this form of wall in the case of a sound backing. The face beneath the arches is laid at the natural angle of slope of the backing. The walls of No. 9 Dock on the Manchester Ship Canal were made in this way, *vide* Fig. 78. Canal walls are essentially the same as quay walls, except that only in certain places will vessels moor alongside the walls, and lighter sections may be used. The same remarks apply to river walls, except where the wall supports a roadway, where the load on that road will have to be taken into consideration in the same way as a quay wall. In some cases the wall is merely to retain and prevent breaking of the bank in floods, and as such works are

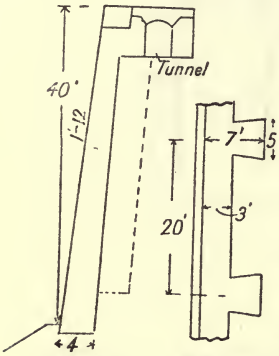


Fig. 76. Fig. 77.

* *Trans. I.C.E.*, Vol. C.

† It would appear that the arches and piers would serve to carry the quay railways and so relieve the wall of thrust due to the moving of heavy loads.

usually constructed by local authorities, and will not bring any monetary return, cheapness is a dominant factor. For

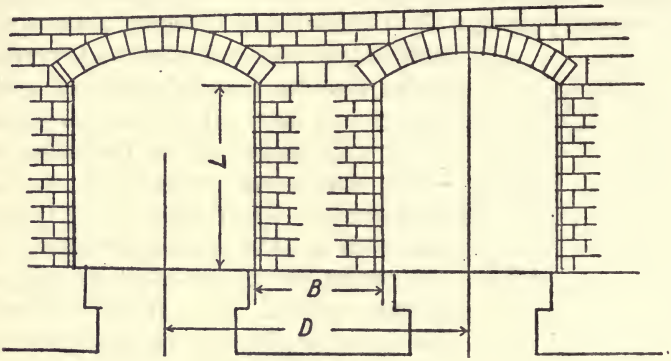


Fig. 78.

this reason the type of wall shown in Fig. 79 is eminently suitable.

Mooring-Rings.

During construction, mooring-rings, bollards, and mooring-posts must be built into the wall. The two latter are generally of cast-iron, of the forms shown in Figs. 80 and 81.

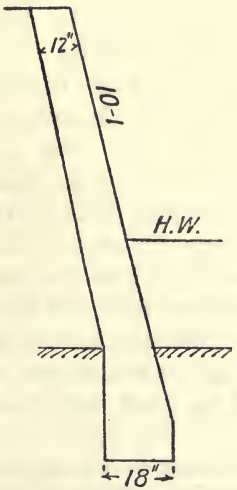


Fig. 79.

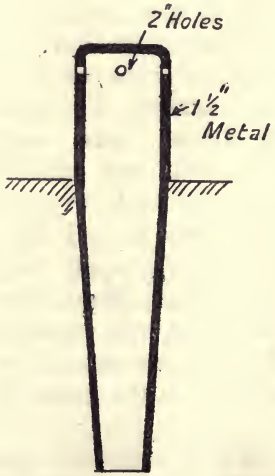


Fig. 80.

PROSPECTUS

PRACTICAL DESIGN OF MARINE SINGLE-ENDED AND DOUBLE-ENDED BOILERS

BY

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wharf surface had sunk nearly 2 ft., and that for a length of about 360 ft. the bulge in the basin-wall had increased from 10 ins. to 19 ins. beyond the true line, the original batter of the face, namely, 1 in 20, being all the while preserved.

"The vessels moored alongside (they were lying 'three off' at the time) were shifted to other berths, and a gang of eighty men set to work TO REMOVE THE BACKING; but by four o'clock in the afternoon it was manifest that the effort to save the wall would prove fruitless, as the bulge had then increased to 3 ft. 2 ins., measuring perpendicularly from the chord of the curve; the men were therefore dismissed for the night to avoid risk to themselves. The wall continued to move outwards until 10.30 p.m., when the central portion of about 200 ft. slid into the dock. In half an hour afterwards the eastern and western parts, of 150 ft. and 100 ft. respectively, followed, turning over within a few minutes of each other. The water in the dock at the time was unusually low, there being but 15 ft. 9 ins. registered on the sill as against 17 ft. 9 ins., the minimum height at neap-tides. By an inspection of Fig. 82 the collapse can be easily accounted for. It is obvious that the foundation was deficient in strength, and that the wall was too thin. The water getting behind, saturated the back filling, reducing the angle of repose to less than 20° (as was subsequently proved by experiment), thus making the back pressure practically hydrostatic, and at the same time reducing the stability due to the dead weight of the wall. The outer toe, where the greatest weight was concentrated, was placed between the first and second rows of piles, and this, bearing on the sleepers, which were originally too thin, and had become more or less unsound, had cut through them, thus giving the final impetus to the wall. It would appear that the second and third portions, not moving quite so far, held for a longer time owing to the support given the sleepers by the second row of piles beneath.

"The history of the portion now under reconstruction is somewhat interesting. The length is about 450 ft., and it was first built in the early part of the year 1849. For a height of 15 ft. from the base it consisted of large flat

stones, weighing from 3 to 5 tons, hammer-dressed on the beds and rough-punched on the face, set flush in strong hydraulic mortar. The upper part, to the level of the coping, was of fair dressed ashlar work, the stones being of good size, backed where shallow with uncoursed rubble masonry. There was evidence that the whole of the masonry work had been carefully executed, and to this is probably to be attributed the fact that it held intact for so many years. Some three months after the completion of the wall, the excavation of the dock was proceeded with, the stuff so removed being used for the back filling. When this backing had reached within 12 ft. of the coping level, the wall was noticed to move outwards to such an extent that it was deemed advisable to take down and rebuild about 200 ft. of it, including the bulged part. An examination of the foundation showed that the outer bearing-piles had canted slightly. The foundation was strengthened by driving a fresh lot of piles."

Sea Walls.—The construction of sea walls and breakwaters is similar to that of dock and quay walls, except that while in the latter case the force of the sea will be almost negligible, in the case of sea walls it is a very dominant factor. Long sloping bulwarks really do not come under the heading of masonry. They have long flat slopes from 3–1 to 7–1 and are made internally of earth or gravel or loose stones, and faced only with blocks of masonry. These blocks should each be capable of singly resisting displacement by the waves, and they take the form of dry stone pitching. Full information on the subject will be found in Mr. Beazley's *Reclamation of Land from the Sea.**

Sea walls are designed similar to masonry reservoir dams, which will be treated later on, and to thoroughly appreciate the construction the action of waves should be considered. Rolling waves in water are propagated horizontally. The motion of each particle takes place in a vertical plane parallel to the direction of propagation; the path or orbit described by each particle is approximately elliptical. The extent of the motion of particles at the water surface,

* Crosby, Lockwood and Co.

both horizontally and vertically, diminishes in inverse ratio to the depth, but the vertical motion will diminish more rapidly than the horizontal one; consequently, when the water is deep, the motion at great depths is quite insensible. The period of a wave is the time occupied by each particle in making one revolution, and is also the time occupied by a wave in travelling a distance equal to its length. The speed of a wave depends on the depth of water and its length. It is greater in deep water and with long waves, except where the depth is greater than the length of the wave, and then the speed will only depend on the length. For all practical purposes the speed of a wave may be taken as equal to the velocity acquired by a falling body whose distance of fall is equal to half the radius of a circle whose length of circumference is equal to that of the wave. In very shallow water, which is usually the case with sea walls, the velocity of the wave is nearly independent of the length, and may be taken as equal to the velocity acquired by a falling body whose distance of fall is equal to half the depth of the water $+\frac{3}{4}$ the height of the wave. When a series of waves advances along a sloping surface, like the seashore, the period of the wave remains constant, but the speed and length diminish and the slope becomes steeper. The result is that the front of each wave tends to become steeper than the back and the crest advances faster than the trough. Consequently the wave *breaks*, a common occurrence to all. Let us take the case of a sea wall with a vertical face, as shown in Fig. 83.*

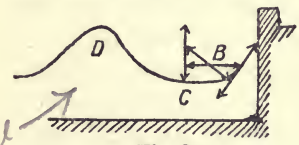


Fig. 83.

If the waves roll straight against this wall they are reflected, and the particles of water for a certain distance in front of the wall have motions compounded of those due to the direct and reflected waves. The result is that the particles in contact with the wall, move up and down through a height equal to *twice* the original height of the waves, and so also do those at C, half a wave-length from the wall.

* Rankine, *Civil Engineering*, p. 755, Twenty-third Edition.

The particles at B, which are one-quarter wave-length from the wall, move backwards and forwards horizontally, and intermediate particles oscillate in lines inclined at various angles. The duty of a sea wall is therefore to reflect waves, which it may do with the face vertical to a slope of 1 to 1. The reflection of waves results in augmenting the height of the waves, and the cope of the wall must be higher than the crest of the highest waves under ordinary circumstances. It is impossible, however, in most cases that the waves will not rise higher than the cope under extraordinary circumstances, and the cope stones of a sea wall should be very large to resist lifting, and be joined to the stones on each side by dowels or cramps, shown in Chapter II, or other methods. The front edge of the cope stones must not project beyond the face of the wall, or else they will be very liable to displacement.

Ashlar and block-in-course masonry are the most usual forms for sea-wall construction, backed with coursed rubble. The whole should be laid in cement mortar and the joints on the face very carefully pointed, so that air and water cannot penetrate the joint; because the continued lashing of a heavy sea would cause water to accumulate in such joints under pressure and displace the stones. The foundation of a sea wall is most important in the way that it will properly resist any undermining action of the sea. The use of groins for this purpose is considered in books on sea defence works and cannot be discussed here, but, so far as the masonry is concerned, the wall should have a good flat toe or pitching, which is not bonded into the wall. Sea walls have been constructed of various sections. The Scarborough sea wall is described as follows:* The sea wall is 3600 ft. long and its face is formed to a curved profile having a radius of 17 ft., the level of the coping being 13 ft. above high-water spring tides. It is founded throughout upon hard shale, being carried down 2 ft. below the surface of the shale. The thickness of the wall is 5 ft. at the top, increasing to 16 ft. at the base, which is at an average level of 9 ft. below high-water

* *Trans. I.C.E.*, Vol. CV.

spring tides. The face of the wall consists of small blocks of Portland cement concrete, composed of 1 of cement to 6 of shingle and sand, the proportions of shingle to sand being about 4 to 2. The blocks were cast in moulds with the face upwards, a chamfer of 1¼ in. in width being formed round the face of each; and the face, for at least 2 ins. in depth, was formed of 2 parts of fine shingle passed through a ¼-in. sieve, and 1 part of Portland cement, no fine sand being used for this purpose. The face of each block was formed at the same time as the block itself, so that the two qualities of concrete should form one substance. The blocks were kept a month before they were set in the wall. In the lower five courses, they were made with a groove formed in the back, 6 ins. wide by 3 ins. deep, and were set in the work so that the grooves should lie transversely to the length of the wall. The object of the grooves was to form a bond with the concrete at the back, and the result proved most satisfactory. The face blocks above the five lower courses were laid in courses, 1 ft. thick, with header and stretcher alternately. The blocks were set in 2 to 1 Portland cement mortar, the joints being well grouted. No machinery was required for setting them, two men being able to carry a block on a hand-barrow, and to lift it into its place on the wall. As the blocks were set, they were backed up to the height of two courses at a time for the full thickness of the wall, with concrete composed of 5½ parts of rough shingle and 2½ parts of sand to 1 part of Portland cement, with pieces of rough stone or rock embedded in it. The coping of the wall, 1½ ft. thick, was formed *in situ* with similar concrete to the hearting of the wall, the top edge being rounded to a radius of 6 inches; and a substantial iron hand-railing was fixed on it. The mixing was done by hand on movable planked staging, as the work was distributed over such a length of ground that the amount of concrete required per lineal foot of wall was only about 4½ cub. yds. The shingle and sand for the concrete were obtained chiefly from the foreshore. The blocks in the face of the wall above high water were not at first provided with a special facing; but at times the foreshore got piled up

against the face of the wall above the level of high water and scoured the surface of some of these blocks, so that subsequently they were specially faced for the whole height of the wall.

Breakwaters are made, to a certain extent, of finished masonry, but the subject hardly comes within the scope of this work. Sometimes they consist of a bank of large blocks of stone placed quite at random up to top water-level and then a parapet wall and coping of the usual description on top. Sometimes they are made of blocks of concrete, and this is now a favourite form of construction.

It may, however, be well to consider such cases of masonry work of which part will be under water, and for the purpose of placing which a cofferdam is impracticable. In that case, soft, unset concrete provides at once an expeditious way of carrying out the work. Such a system was adopted by Mr. William Kidd at Blyth Harbour,* and subsequently at Skinningrove, Yorkshire, for the Skinningrove Iron Company. Mr. Kidd says :†

" The pier was constructed in lengths of 20 ft., without staging. A 5-ton steam crane, advanced to the end of a completed length, commanded the next one. The sand was first removed by an 8-in. centrifugal pump with two 6-in. suction nozzles ; one of these was connected with an india-rubber hose, and, being guided by a diver, cleared the sand from a space 25 ft. long by 40 ft. broad, laying bare the underlying shale ; the other nozzle meanwhile sucked up water to keep the pump from choking. The sand stood well at a steep angle under water at the sides of the excavated space ; and, moreover, the concrete was deposited immediately the bottom was cleared. The freshly-mixed concrete was deposited from hopper boxes, holding 20 cub. ft., lowered by the crane, and directed by a diver, so as to make a ' reef ' over the entire excavated space, without any protection, and forming natural slopes at the sides of about 1 to 1. It was levelled and trodden down. The concrete was composed of $2\frac{1}{2}$ parts of broken slag, $\frac{1}{2}$ of

* *Trans. I.C.E.*, Vol. LXXXI.

† *Trans. I.C.E.*, Vol. CV ; and *Engineering*, November 3rd, 1890.

rough gravel, 1 of sand, and 1 of cement, measured separately. The formation of the 'reef' was carried on continuously, from the first deposit of concrete, until the whole excavated space forming one length of foundation was covered, and raised above the level of the surrounding sand. It was then completed the following day, or as soon as the weather permitted. A length of about 300 lineal ft. of the lower part of the North Pier has been executed in this manner, including also an apron along its seaward side.

"The use of large bags or blocks is almost universal for

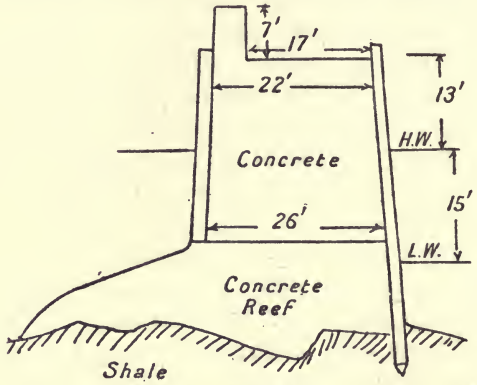


Fig. 84.

the portions under water of the exposed parts of harbour works along the north-east coast; and apparently no attempt had been made to use concrete in mass exclusively for this purpose. A trial length, however, carried out as described, afforded satisfactory results; and experience during the subsequent progress of the work established the success of the system."

A section of the wall is given in Fig. 84. The curved section of sea wall is a very favourite one with engineers. In some cases the wall is made as shown in Fig. 85, with a slightly overhanging cope stone. This serves to throw off the waves from the wall when they rise, but the amount of overhang must be strictly limited or else the waves will collide with, rather than be deflected by, the coping and the

stones will be displaced. Many sea walls have simply a slanting face. The form of cross-section is not of much importance, but the foundation is. This should be preferably in the form of a toe, 4 or 5 ft. wide, of rubble in cement mortar laid behind sheet piling, which may be left in. It serves to keep the actual foundations from being undermined by the sea. It is best put in with the wall, but in case of subsequent signs of undermining may be put in afterwards.

Berms.

The method of constructing a "berm" has much to recommend it, because it allows sea-wall work to proceed during the winter, which is almost essential in seaside resorts. By it every short length of face work can be made separately secure and the joints can be bonded horizontally and vertically as the work proceeds. The projecting base will serve as a promenade, but its great value lies in the fact that heavy rollers are broken up in two stages, so that only the crest comes against the wall proper, the base taking the weight of the attacking waves, and the quantity of overthrow on to the promenade proper is quite small, even in rough weather.

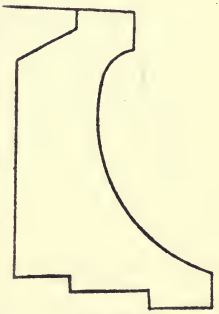


Fig. 85.

It has a disadvantage in that the scouring action of the beach is increased, but even this can be prevented by proper groins. In constructing the face of a masonry sea wall the joints must be broken *horizontally* and *vertically*, because if a wave attacks a wall at right angles to its motion it will expend its force in a horizontal line throughout the wall, and if the stones are laid in continuous unbroken courses the strain is unevenly distributed, being confined in intensity to one horizontal joint. The safety of the wall depends to a great extent upon the weight of the structure, and the efficiency of the jointing, and in consequence, if the horizontal joints are broken, it is impossible for any one single course to be carried away without separating those below.

In sea-wall work at Penzance Mr. Latham adopted this principle, as shown in Fig. 86.*

The courses are laid with headers and stretchers, as shown, with ashlar granite face, battered 3 ins. per foot vertical. The courses are bedded and jointed in mortar, consisting of equal measures of Portland cement and clean, sharp sand, and one-ninth of a measure of trass; and the work is pointed in neat cement. Above the foundation the facing ashlar is reduced 6 ins. in thickness from front to back, the stretchers are 12 ins. deep, and the headers 1 ft. 6 in. Every vertical bonding stone or "jumper" is of additional depth, in order to afford strength to the horizontal joints and extra tie into the concrete. For a distance of 2 ft. from the back of the ashlar facing the work is packed solid with specially gauged concrete, consisting of 2 parts sharp quarry stone siftings, ranging from ½ in. down to the size of a pin's head, and 1 part small beach gravel about the size of peas, and 1 part sharp beach sand; to this was added 1 measure of Portland cement, and $\frac{1}{9}$th of a measure of trass. The same kind of material is placed against the face of the old wall for 1 ft. in thickness.

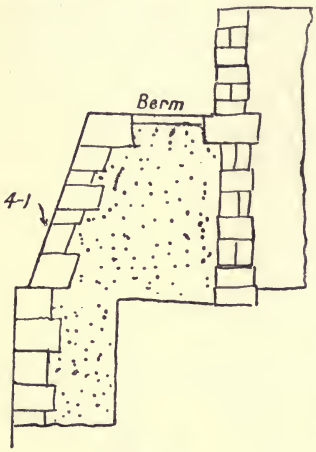


Fig. 86.

The remaining space between the two walls is made solid with concrete, gauged 4 measures of beach gravel and sand, mixed in proper proportions, and 1 of Portland cement. In this "fig" stones were allowed to be bedded 6 ins. apart; they were required to be rough granite spalls of irregular shape; no round cobbles or other smooth stones were admitted.

Coping.—The coping course, being exposed to considerable danger, received careful consideration. The facing

* *Trans. I.C.E.*, Ireland, Vol XXXII.

course immediately below the coping is made 6 ins. deeper than the rest of the headers and stretchers, the jumpers breaking joint vertically and being proportionately large. This course was solidly packed with mortar and concrete. At intervals of every 3 ft. a vertical 1¼-in. iron bolt, with head at top and nut and washer at bottom, was bedded 12 ins. in the concrete, behind the facing stones. Each cope was made to break joint with the face work and also with the granite surface paving, 8 ins. thick, and the backs of the cope stones were tooled to this depth, while the remainder was rough splayed at the back in the form of a foot to be embedded in the concrete backing, so that the stone, if removed, would have to tear away the concrete and lift the paving. The end of each stone is provided with a vertical joggle, consisting of a dovetail chase 3 ins. deep and 2 ins. wide at the gap, and 3 ins. at the base. The cope stones were bedded in mortar and the joints caulked with sackcloth, and neat cement grout poured into the dovetails, which met so ⋈ on adjacent stones. The cost of this class of work, as executed by Mr. Latham, is appended below :—

The Cost of Construction with Heavy Blocks.

1000 cub. yds. of ashlar masonry in heavy blocks, in cement mortar, at £1 15s. ..	£1720	0	0
323 sq. yds. granite paving, at £1 5s. ..	403	15	0
1200 cub. yds. of excavating, at 4s.	240	0	0
15 ,, of steps, at £3	45	0	0
731 ,, of concrete, at 16s.	584	16	0
30 ,, of granite tie stones, at £1 10s.	45	0	0
Hacking out present joints, and cutting out a course of masonry for uniting the new and old work	43	0	0
Timbering trenches	20	0	0
Cranes and pumping charges	350	0	0
Clerk of Works, 24 weeks at £2	48	0	0
Contingencies	300	0	0
	£3799	11	0

Another sea wall of masonry which is of interest is that on the Chester and Holyhead (L.N.W.R.) Railway, originally built by Mr. Robert Stephenson, and it is one and a quarter miles long. It is shown in section in Fig. 87. The curved parapet was designed to throw the water off the rails, and is 8 ft. above rail level and has a radius of 25 ft. The wall is of dressed ashlar and coursed rubble. Sea walls, of masonry or concrete, usually take a form similar to those described, that is to say, the sloping or curved wall is preferable to a vertical one. This is always the case in shallow water, but, as will be shown in the next paragraph, and as has been shown in the case of quay walls, that for deep water a vertical face, or nearly so, is adopted. We are not concerned with earthen embankments for sea walls, they are merely faced with dry stone pitching and do not fall under the heading of masonry, and the subject of breakwaters will be briefly considered. A breakwater in the true sense of the term cannot be considered as true masonry. Constructed principally of stone it is true, but the stone would be simply blocks quarried in any shape and deposited out to sea in the form of a bank till it rose above the highest tides. When solid, along its top a masonry wall or parapet and cope may or may not be constructed. In some cases, however, the work is partly of masonry below water. Such work would generally be executed by means of a diving-bell. The vertical wall at the top of a sloping stone back transforms the motion of a wave into a horizontal one, the velocity of which will tend to scour the foundations and will cause the displacement of large blocks of stone, and has in consequence been only reservedly adopted. Long rubble slopes, 7 to 1 say, on the other hand, are waste of money and also bad construction, inasmuch as it is exposed to direct percussive action

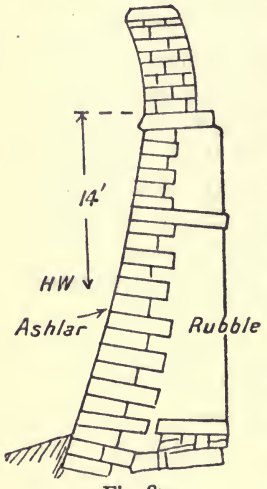


Fig. 87.

of the waves and "undertow," or recoil, of the sea, and unless very heavily pitched such slopes will suffer; 1 to 1 or 1½ to 1 has been found less costly and more satisfactory as a general rule in practice. In depositing the

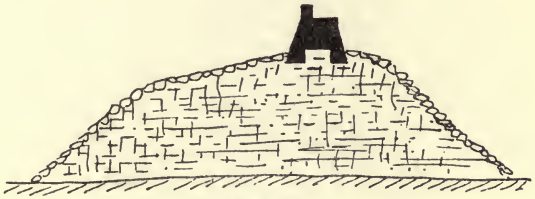


Fig. 88.

stone foundation the larger stones should be reserved for the top of the wall, diminishing in size downwards, as the action of the sea decreases in intensity as the depth. In fact, large blocks of concrete have been simply deposited for this

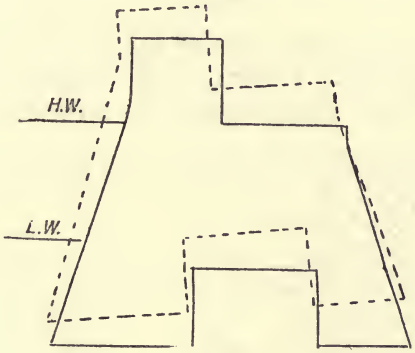


Fig. 89.

purpose and have given good results, blocks of as much as 140 tons being used by Mr. Stoney on the Poolbeg wall in Dublin Harbour. Alderney Breakwater* is an example of one on which a masonry top was placed on a deposited foundation. The general profile is given in Fig. 88, and two forms of masonry superstructure in Figs. 89 and 90. That in Fig. 89 settled as shown, and in consequence the rest of the work was executed as shown in Fig. 90, the

* *Vide Trans. I.C.E.*, Vol. XXXVII.

facing being granite headers 3 ft. thick, some of which were joggled together and dowelled. The backing was 30 ft. of concrete blocks and the rest of stone in cement mortar.

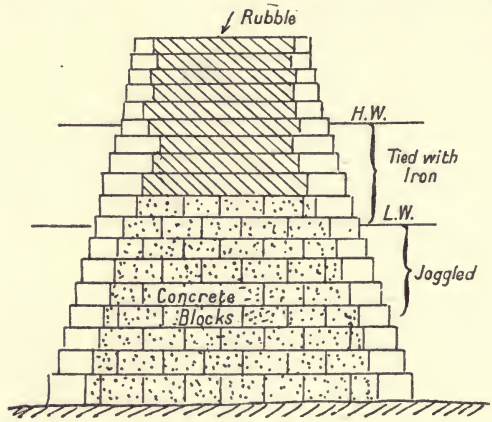


Fig. 90.

Some of the quoins were secured by iron bars and diagonal straps bolted to the masonry. In the case of Holyhead Breakwater,* the superstructure is of the form shown in

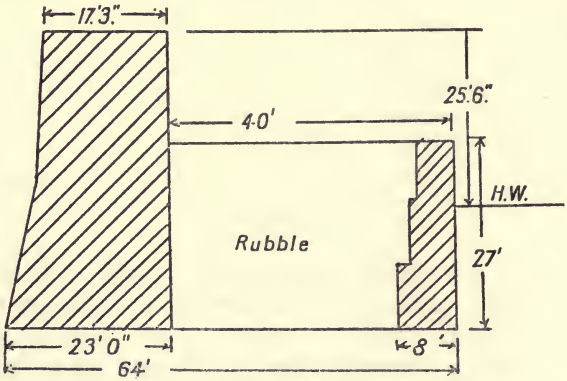
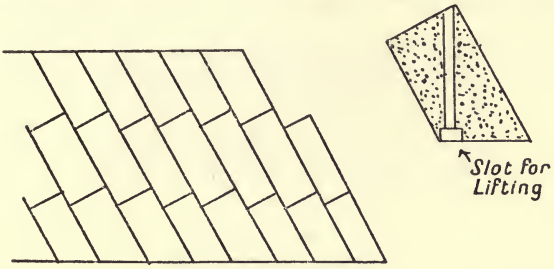


Fig. 91.

Fig. 91, and is to prevent deposit washing over into the inner harbour. There is also an inner wall with a vertical face. The hearting is rubble. The wall cost £489 per lineal yard.

* *Trans. I.C.E.*, Vol. XLIV.

Concrete blocks are now largely used for breakwaters owing to the size in which they can be made and the ease of deposition. It is a very favourite construction abroad.



Figs. 92 and 93.

The blocks are made in the form shown in Figs. 92 and 93, and when deposited by a Titan crane, running on rails laid down on the finished work, take the set shown. The system has been employed at Kurachee* and other places, notably Madras and Kustendji.†

This form of superstructure can settle without permanent dislocation; 30 tons is a suitable weight for the blocks. On the Amsterdam Canal similar blocks were used for the North Sea pier, but the courses were horizontal. Mr. B. B. Stoney ‡ says that this form of construction saves time and reduces cost, and where a wide top is required the outside may be of one or two rows of blocks and a rubble hearting. An upright or slightly battered face is generally employed for this class of work.

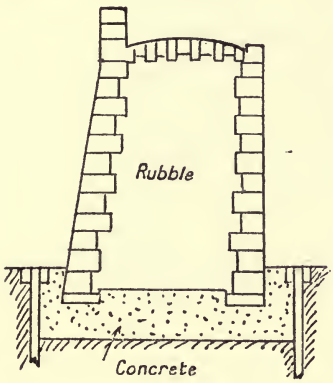


Fig. 94.

Piers.

Masonry piers are executed usually with a hearting of rubble masonry or concrete and an ashlar facing, but the

* *Trans. I.C.E.*, Vol. XLIII. † *Ibid.*, XXXIX. ‡ *Ibid.*, XXXVII.

filling may be earthwork if the two side walls are tied by cross-walls. Very large stones should be used. A suitable section is shown in Fig. 94, and if filled with earth, Fig. 95. The foundations should be of the most substantial form,

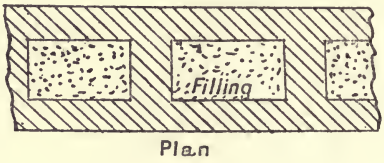


Fig. 95.

either of concrete, as shown in Fig. 94, or piles and sheet piling; and, in the case of a subsoil easily scoured, an apron of stone, held in place with piles, must be formed on each side of the wall, especially at the head. The stone would be simply deposited. Of course, the execution of coursed

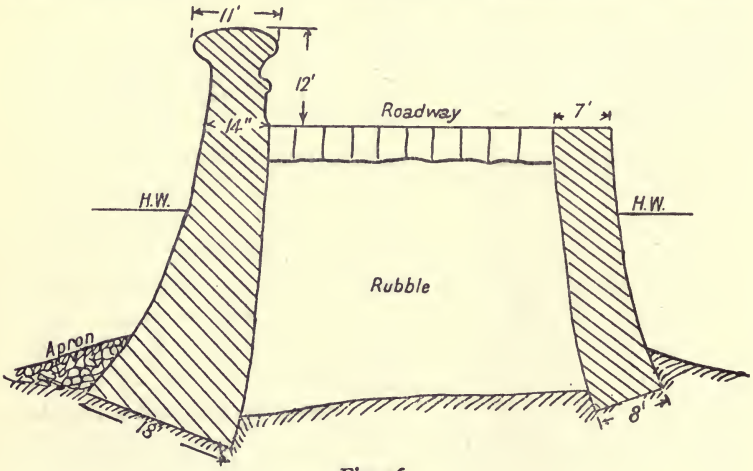


Fig. 96.

masonry below water entails either a cofferdam or caisson, and in deep water this is almost impossible; but, on the other hand, the rubble foundation in deep water is likely to be more secure than in shallow, and will, in fact, become very solid by the filling up of the interstices with deposit. At Whitehaven the West Pier (Fig. 96) is a true masonry

wall, and was constructed by Rennie on a soft sandstone foundation. The walls are ashlar with rubble filling. The roadway is 3 ft. ashlar. The curved face has been found to prevent water being thrown on to the road, except in very rough weather. An apron is formed as shown. The pier at Dover (old works) (Fig. 97) was put in with diving-

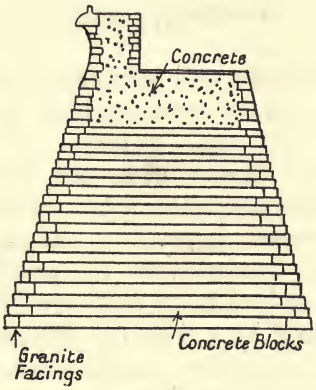


Fig. 97.

bells and cost as much as £415 per foot. The facing only is of granite, and the body of the wall is solid concrete blocks truly rectangular set and bedded like stone. The upper portion is monolithic concrete. Many examples of sea walls, etc., could be given, but as the principles of construction are the same in all cases the reader is referred for information on executed works to the engineering papers, and especially to the *Transactions of the Institute of Civil Engineers.*

CHAPTER V

MASONRY DAMS

RESERVOIR dams are frequently built of masonry, and this form of construction is now becoming widely adopted for this purpose for dams of any height. The actual dam is, of course, simply a retaining-wall for water whose pressure is taken as being concentrated at the centre of pressure in precisely the same way as before described. Owing, however, to the great height of such dams, and compared with an ordinary retaining-wall, the design is considered in a slightly different manner, and there are also considerable modifications of form. The latter is due to the fact that by going into minute details in regard to a dam of any considerable height masonry may be saved by reducing the cross-section without reducing the strength; and when it comes to a long dam, the saving of a few feet in cross-section means many cubic feet in the end. For instance, take a dam whose sloping face measures 120 ft. If the thickness of the dam is uniformly reduced 1 in. only this means $120 \times 12 \times 1 = 1440$ sq. ins. $= 10$ sq. ft., and if the dam was 1000 ft. long it would mean 10,000 cub. ft. of masonry saved, a considerable saving in cost.

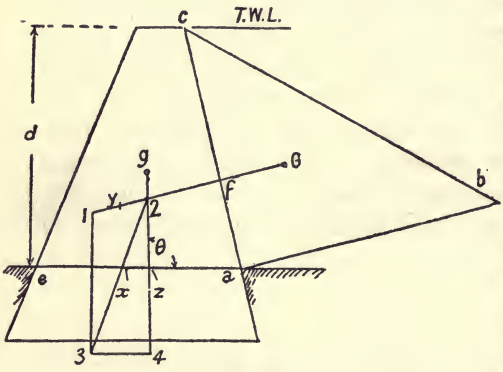


Fig. 98.

Take any dam of trapezoidal section, as shown in Fig. 98,

on which the side $a\,c$ is exposed to water pressure, whose top water-level is denoted by T W L. Consider a layer of the water in length equal to 1 ft. Draw $a\,b$ at right angles to $a\,c$ and equal to d and complete the triangle $a\,b\,c$. This is, then, the triangular prism of water whose weight, at $62\frac{1}{2}$ lbs. per foot cube, will be equal to the total pressure exerted against that layer of the wall. If G is its centre of gravity and G f is perpendicular to $a\,c$, the pressure due to the water acts in this direction. In the same way, if g=the centre of gravity of the wall, the pressure due to the weight of the wall acts vertically from g downwards, and consequently the parallelogram 1 2 3 4 can be drawn by making 1–2 equal to the water pressure and 2–4 equal weight of wall. When complete, the diagonal 2–3 is the resultant which must fall within the middle third of that wall. Now in any dam let d be as shown, =depth of water, and w=weight of 1 cub. ft. of water=$62\frac{1}{2}$ lbs., while ϕ= inclination of $a\,c$ to the vertical. The pressure due to the vertical layer of water is

$$\mathrm{P}=\frac{wd^2}{2}\sec\phi \quad . \quad . \quad . \quad (23)$$

and its centre of pressure is $\frac{2}{3}$ below the surface. This force P combined with the equal and opposite oblique component of the resistance of the joint at x constitutes a couple, tending always to overturn the wall. The arm of the couple is equal to the perpendicular distance xy

or $$fa-xa\sin\phi \quad . \quad . \quad . \quad (24)$$

But $$fa=\frac{d\sec\phi}{3} \quad . \quad . \quad . \quad . \quad (25)$$

Call ea, $t\,\therefore xa=t(q+\cdot5)$ when q is equal to or less than

$$\frac{w^1}{3w}\cdot\tan^2\theta-\tfrac{1}{6} \quad . \quad . \quad . \quad (26)$$

Then for the arm of the couple in question we have

$$\frac{d\sec\phi}{3}-t(q+\cdot5)\sin\phi \quad . \quad . \quad . \quad (27)$$

If this is multiplied by the pressure the overturning moment is the result. This must be less than the moment of stability

of the wall in practice (and in theory at least equal to it); consequently the equation as follows holds good if W= weight of unit length of wall.

$$W \times xz = W(q \pm q_1)t = \frac{wd^3}{6} \sec^2 \phi - wd^2t\left(\frac{q}{2} + \cdot 25\right) \tan \phi \quad . \quad (28)$$

If the wall has a vertical face inside $\sec \phi = 1$ and $\tan \phi = o$, and the equation may be simplified to the form

$$W(q \pm q^1)t = \frac{wd^3}{6} \quad . \quad . \quad . \quad (29)$$

If a wall of any section requires to be dealt with circumscribe a rectangle about it and find its cross-sectional area and call the ratio of the sectional area of the actual wall to the rectangular one r. Then the weight of the vertical layer of masonry under consideration is

$$W = w^1 r dt \quad . \quad . \quad . \quad (30)$$

where w^1=weight of 1 cub. ft. of masonry, d and t being in feet. In this case the above formula will become modified to the following form:—

$$r(q \pm q^1)w^1 dt^2 = \frac{wd^3}{6} \sec^2 \phi - wd^2t\left(\frac{q}{2} + \cdot 25\right) \tan \phi^* \quad . \quad (31)$$

and

$$r(q \pm q^1)w^1 dt^2 = \frac{wd^3}{6} \quad . \quad . \quad . \quad (32)$$

We can now compute the thickness of the wall t at any point after having assumed it. Referring to Equation 31, let

$$a = \frac{wd^3 \cdot \sec^2 \phi}{6r(q \pm q^1)w^1 d} \quad . \quad . \quad . \quad (33)$$

$$\beta = \frac{wd^2(\frac{q}{2} + \cdot 25) \tan \phi}{2r(q \pm q^1)w^1 d} \quad . \quad . \quad (34)$$

Then $$t^2 = a - 2\beta t \quad . \quad . \quad . \quad . \quad (35)$$

or $$t = \sqrt{(a + \beta^2)} - \beta \quad . \quad . \quad . \quad (36)$$

But in the case of a vertical wall $\beta = o \quad \therefore t = \sqrt{a}$.

The vertical component of the water pressure is

$$P \sin \phi = \frac{wd^2}{2} \tan \phi \quad . \quad . \quad . \quad (37)$$

* These equations assume the water-level as shown in Fig. 98. If below the crest use h=height of wall in place of d in first part of equation, but d is generally used.

and the horizontal component

$$P \cos \phi = \frac{wd^2}{2} \quad . \quad . \quad . \quad . \quad (38)$$

The stability of friction at any joint such as

$$ea = \frac{P \cos \phi}{W + P \sin \phi} = \frac{wd^2}{2W + wd^2 \tan \phi} \leq \tan \theta - 90° \quad (39)$$

when $\theta - 90° =$ coefficient of friction $= \cdot 74$.

Again, let $$\frac{a}{d^2} = A \text{ and } \frac{\beta}{d} = B \quad . \quad . \quad . \quad (40)$$

Then $$\frac{t}{d} = \sqrt{A + B^2} - B \text{ and } W = rw^1 dt = rw^1 d^2 \frac{t}{d} \quad . \quad (41)$$

The common factor d may be cancelled, and then

$$\frac{P \cos \phi}{W + P \sin \phi} = \frac{w}{2rw^1 \frac{t}{d} + w \tan \phi} \leq \tan \theta - 90° \quad . \quad (42)$$

The equations for stability are usually neglected, as they are generally fulfilled for the greatest possible practical values of q. In the case of a triangular wall having no thickness at the apex, the ratio $\frac{t}{d}$ is constant at every joint, so that only one joint need be tested for stability, and, in consequence, a triangular section is uniformly stable. The value of r when inner face is vertical

$$\cdot 5 \; q^1 = \tfrac{1}{6} \; \phi = o \quad \therefore \frac{t}{d} = \sqrt{A} = \sqrt{\left[\frac{w}{3(q + \frac{1}{6})w^1}\right]} \quad . \quad . \quad (43)$$

$$\therefore \; w^1 \frac{t}{d} = \sqrt{\left\{3(q + \tfrac{1}{6}) \frac{w}{w^1}\right\}} \leq \tan \theta - 90° \quad . \quad (44)$$

Assuming q as given previously

$$\frac{t}{d} = \frac{w}{w^1 \tan \theta} \quad . \quad . \quad . \quad (45)$$

In the case of a rectangular wall, which is not a usual section, but simply given for comparison, $r = 1$ q^1 and $\phi =$ zero.

$$\therefore A = \frac{w}{6qw^1} \text{ and } B = \text{zero} \quad . \quad . \quad . \quad (46)$$

The ratio $$\frac{t}{d} = \sqrt{A} = \sqrt{\frac{w}{6qw^1}} \quad . \quad . \quad . \quad (47)$$

and equation for stability not usually required

$$= \frac{w}{2w^1\sqrt{\frac{w}{6qw^1}}} = \sqrt{\frac{3qw}{2w^1}} \angle \tan\theta. \quad . \quad (48)$$

Triangular walls are often made with the axes vertical, especially in the case of a soft foundation. This will ensure the centre of resistance x being at the middle of each joint and vertically below the centre of gravity of the part of the wall above the joint. The point 2 in the diagram will also fall in the middle of each joint. These conditions, however, will only hold good when the section is that of an isosceles, the outer and inner faces forming equal angles ϕ on opposite sides of the vertical axis of the wall. In other words, Gf should bisect the base so that

$$\frac{t \sin\phi}{2} = \frac{d \sec\phi}{3}$$

Now $\tan\phi = \frac{t}{2d}$ $\therefore \sin^2\phi = \cdot 33$ and $\cos^2\phi = \cdot 66$. (49)

consequently the value of

$$\tan\phi = \frac{t}{2d} = \sqrt{\cdot 5} = \cdot 707$$

consequently the base of the wall bears the same ratio to its height as the diagonal of a square does to its side. In this case

$$\frac{w}{w^1\sqrt{2} + w\sqrt{\cdot 5}} = \frac{w\sqrt{2}}{2w^1 + w} \angle \tan\theta \quad . \quad . \quad (50)$$

It was stated, however, that this type of wall is suited to a soft foundation by giving a uniform pressure over this foundation in consequence. The value of $\tan\theta$ must be the value for the material on which the dam will rest, and may be about one-quarter in value. But the masonry must have a specific gravity in excess of a certain amount. The equation as follows holds good, that is, $w^1 \angle 2(\sqrt{2} - \tan\theta)\, w$, or if $\tan\theta = \frac{1}{4}$ as stated, the weight of w^1 = weight of masonry per foot cube $= 2 \cdot 33w = 2 \cdot 33 \times 62 \cdot 5 = 145$ lbs., otherwise the dam will not resist the tendency to slide on any foundation having any value of $\tan\theta$ by friction alone.

Construction.—The material of which a dam is made de-

pends to a certain extent upon the nature of the foundations. For masonry dams rock is essential, except in small dams, because otherwise the width of foundation necessary to get a safe intensity of pressure on the earth forming them will be very much greater than the necessary width calculated by the foregoing or following considerations. Rubble masonry is in all cases preferable to regularly coursed masonry for dams, as the irregularity of the joints gives a far greater protection against leaks, owing to the tortuous course any leak would have to take if started. The stones should be carefully dressed, however, and bedded on cement mortar. All spaces should be filled with stones, no matter how small, and no grouting should be allowed under any circumstances. The actual cross-section will primarily depend on the rule that the intensity of pressure on the inner face with the reservoir empty and on the outer face when full shall at no point exceed a certain safe limit. This limit should be lower in the outer than in the inner face, and the generally accepted limits are 9 tons per square foot on the inner and 7 tons per square foot on the outer face. These limits have, however, been exceeded in practice in dams (e.g. Lake Vyrnwy, Liverpool Waterworks) which have never given any sign of failure. The reason for the lower limit on the outer face is that the pressure acts at a tangent to both faces and the outer face is usually far removed from the vertical, consequently the ordinary formulæ do not give the true intensity of pressure, but only the vertical component, which is less than the actual in the same ratio as the face departs from the vertical. The inner face has generally a small batter, but should not have any overhang. It was originally thought that an overhang inside would increase the stability by increasing the moment opposite to the overturning, but it must always be borne in mind that half the weight of the overhang (approximately) will be lost due to buoyancy in the water and, in consequence, the material is not distributed to a good advantage in such a position, and the effect it may have will be quite out of proportion to the expense. Tension on the inner face must be avoided

at all costs by the line of resistance falling in the middle third of the base and all sections. A slight curvature in plan, without being considered as allowing the dam to depart from a true gravity dam, is an advantage.

Rankine Method.—In the general form of masonry dam, adopted by Professor Rankine, the inner face has a slight batter. The design of this type of dam is based on the fact that at any depth, d_1, below the surface equal to 120 ft. the thickness T=84 ft. The curves of the faces are logarithmic, and a constant subtangent to the two is given by Rankine as 80 ft.=a. Then, knowing these quantities, the design of any dam can be proceeded with as follows: At any depth d, the thickness t=

$$\log t = \log \mathrm{T} + \cdot 4343 \frac{d-d_1}{a} \quad . \quad . \quad (51)$$

The top thickness is assumed as 18·74 ft.=t_0. Now the sectional area of any section of the wall, measured from the top of the wall=depth D.

$$\mathrm{A} = \int_0^d t\mathrm{D}d \quad . \quad . \quad . \quad (52)$$

$=a(t-t_0)$. The lines of resistance for reservoir full and empty may now be drawn on any dam section.

They pass through the centre of gravity of any part of the wall above a given plane. Now, as before stated, the moment of horizontal pressure of the water at any point on the wall at depth d=

$$\mathrm{M} = \frac{d^3}{12} \quad . \quad . \quad . \quad . \quad (53)$$

Find the cross-section area of the wall above any given depth d and call it A.

Then

$$\frac{d^3}{12} \div \mathrm{A} \quad . \quad . \quad . \quad . \quad (54)$$

gives the horizontal distance x between the lines of resistance for reservoir full and reservoir empty, in other words,

$$\frac{\mathrm{M}}{\int td^3} = \frac{\mathrm{D}d}{12a(t-t_0)} \quad . \quad . \quad . \quad (55)$$

As the inner side of the dam has a sloping face there will

be a certain amount of vertical pressure due to the water which will add to the stability of the wall, but in design it is so small as to be practically negligible. Now, on any plane D feet below the top

$$p=\int\frac{tDd}{t}=a\left(1-\frac{t_0}{t}\right) \text{ or if R= } . \quad . \quad (56)$$

deviation of line of resistance from middle of any horizontal plane the greatest value of

$$p=a\left(1-\frac{t_0}{t}\right)\left(1+\frac{6R}{t}\right) \quad . \quad . \quad . \quad (57)$$

when R=the above deviation.

Low Dams.—The above form of dam is not suited to small depths, as it gives for such depth an uneconomical distribution of material, and other formulæ are resorted to. The depth under which they are unsuitable is 110 ft., and in designing a dam the most economical section will result from employing the above for that part of the dam above 110 ft. high and the following below this. In Fig. 99 we see that the width of base consists of two quantities, A and B. Then if we decide upon the height H in feet, and P=the maximum pressure on the masonry in lbs. per square foot, the value of B is

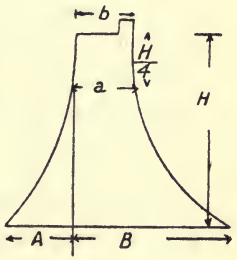


Fig. 99.

$$\sqrt{\frac{\cdot 05H^3}{P+\cdot 03H}} \quad . \quad . \quad . \quad (58)$$

And again

$$A=\left(\frac{\cdot 09H}{P}\right)^4 \quad . \quad . \quad . \quad (59)$$

But the minimum value of B should in no case be less than ·6H. To get the curved face of the wall as shown, simply find a number of values of B corresponding to the particular value of H. A considerable saving of material is thus made. When we get to near the top we set off a distance $=\frac{1}{4}$H as shown, and to find the value of *a* Equation 58 is used, H being only $\frac{1}{4}$ of its maximum value. At the same time we make $b=\cdot 4a$. Regarding the safe pressure on the

masonry, if it is light work 6 tons is ample to the square foot, but in heavy work it may with safety be increased to 9 tons. The work is usually done in uncoursed squared rubble, the foundations (of solid rock if possible) being carefully checkered out to form a key, in the same way as for embankments. Fig. 100 shows another design for a large reservoir wall. In this case

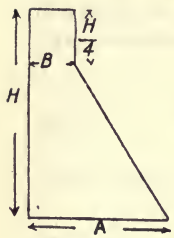


Fig. 100.

$$\left.\begin{aligned} B &= \cdot 18H \\ A &= \cdot 72H \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (60)$$

based on the assumption that the masonry weighs 110 lbs. per foot cube.

Arched Dams.—Now some engineers have adopted, with more or less success, a curved arched form for reservoir walls, the crown of the arch facing up stream. Of course, we can, and often, see such walls having a small curve, perhaps for various minor considerations, which the engineer had in mind when designing. But when certain calculations are gone into, by using such a form of wall in plan, we can often effect a saving of material. Of course, the engineer has to find out the extent of such a saving. There are three good existing practical examples of such dam :—

1. At Aix, in France, *the Zola dam.*
2. In California, *the Bear Valley dam.*
3. In California, *the Sweetwater dam.*

It is, however, owing to the present-day imperfect knowledge of engineers as to what happens in the internal parts of a dam under stress, and to the necessary higher average intensity of such stress in dams that are curved, which has prevented their more general adoption. Let R=the radius on water face and r=the outer radius of the proposed dam at any point on the same level; then, by the theory of arches when w=weight of 1 cub. ft. of water=62½ lbs., and if R and h are, as in Fig. 101,

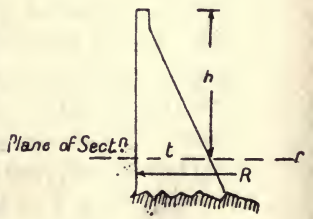


Fig. 101.

the horizontal thrust at every point of that particular curve

$$=Rhw. \quad . \quad . \quad . \quad (61)$$

If now the maximum stress on the masonry is limited to 10 tons per square foot it will be obvious to the reader that the thickness of the dam at the point in question

$$=R-r=t \text{ feet.}$$

And
$$t=R\left(1-\sqrt{1-\frac{2hw}{S}}\right) \quad . \quad . \quad . \quad (62)$$

Say R=100 and h=100, w=62·5 and S=10 tons per square foot. Then

$$t=100\left(1-\sqrt{1-\frac{2\times 100\times 62\cdot 5}{10\times 2240}}\right)$$

$$=100(1-\sqrt{\cdot 444})=100(1-\cdot 657)$$

$$=100\times\cdot 343=34\cdot 3'=t.$$

This formula may be applied to the Garrett dam shown later.

Buttressed Masonry Dams

Many attempts have been made to build dams in such a way that less material may be used by so distributing the masonry that it shall be fully stressed in all parts. This can in theory be successfully done and has been done in practice; at the same time, it is well to note that engineers, so far as British practice is concerned, are content to adopt the gravity type still, rather than run undue risks with new types. Mr. Sandeman, the engineer of the Derwent Valley works, which at the time of writing are the latest examples of large masonry dams, has been faithful to the gravity dam. To the author's mind there is an element of caution with British engineers bordering on timidity, and in consequence the following suggestions should be worthy of the attention of the younger members.

The buttressed masonry dam provides a type which is thinner than a gravity dam and in which the thrust is

taken by buttresses. The inner face, as well as the outer one, has a batter.

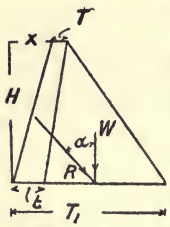


Fig. 102.

The first procedure is to settle upon the batter which will be given to the outer face. Referring to Fig. 102 the value of x is required. Let T, t, T_1, H, and x be as shown, while let the ratio of the clear space between the buttresses to the thickness of those buttresses be denoted by r. Then a general formula to find x will stand

$$x=\frac{T_1^3+rT^3}{T_1^2+rt^2}-S\left(\frac{T_1+rt}{2r+1}+T\right) \quad . \quad . \quad (63)$$

S being the specific gravity of the masonry, usually from 2 to 2·5. The value of x so found is the minimum safe value. The values of T, t and r have to be provisionally settled by the designer, the most suitable being—

$$T=\cdot 5+\frac{H}{10} \quad . \quad . \quad . \quad (64)$$

$$t=T+\frac{H}{20} \quad . \quad . \quad . \quad (65)$$

$$r=\frac{50-H}{10} \quad . \quad . \quad . \quad (66)$$

while the thickness B of the buttress may be taken as 2 ins. for every 1 ft. in value of H. There is an equation for T_1, but it is of a very complex nature, and its discussion would be out of place in a book of this sort, and certain values are tabulated as under :—

Value of H.	Value of T_1.
5	5·90
6	6·80
7	7·90
8	9·00
9	10·00
10	11·40
11	12·50
12	13·55
13	14·70

Value of H.	Value of T_1.
14	15·90
15	17·00
16	18·25
17	18·75
18	20
19	22
20	23·25
21	24·50
22	25·75
23	27·00
24	28·50
25	30·00
26	31·25
27	33·00
28	34·50
29	36·00
30	38·00

When the above values are ascertained for a given value of H, it will be necessary to prove if the resulting structure will have the maximum pressure at the toe within safe limits, and also that the resultant does not make an angle with the vertical greater than the angle of repose. Referring again to Fig. 102, it is required to find the angle a and also the value of P, the maximum pressure, at the toe as follows :—

$$\cot a = \frac{(r+1)(x+ST)+S(T_1+rt)}{H(r+1)} \quad . \quad . \quad (67)$$

$$\text{and } P = \cdot 0280\, H^2(r+1)\frac{T^1 \cot a \sec^2 a}{T_1{}^2+rt^2} \quad . \quad (68)$$

and having found cot a we have another equation for x which may be used as a check ; that is—

$$x = H \cot a - ST - \frac{S(T_1+rt)}{r+1} \quad . \quad . \quad (69)$$

If the value of a is limited to 35°, which is the angle recommended by Professor Rankine, the value of x will increase, but the writer is inclined to believe that a may be as much

as 42° and the wall quite safe. The tension in the dam between the buttresses can be easily computed by assuming it as a loaded and supported beam (not fixed) between the buttresses, which would be placed 6 to 8 ft. apart according to circumstances. No tension will exist if the inner face between the buttresses is slightly arched on plan. When such a dam has been designed it remains with the designer to calculate its cubic contents, and also the cubic contents of a solid dam to fill the same conditions. The saving is generally about 35 per cent in actual masonry, but it remains with the judgment of the engineer if the actual saving in cubic feet of masonry is going to counterbalance the extra cost entailed in building a wall of this more complicated description. The weight of the wall between the buttresses will be found as follows. Referring to Fig. 103, Z being as shown and $w=62\frac{1}{2}$ lbs. per foot cube

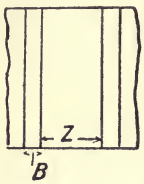


Fig. 103.

$$W_1=Z\times\frac{T+t}{2}\times HSw \quad . \quad . \quad (70)$$

and the overturning moment due to the water is

$$M=\frac{H^3}{6}(r-1)Bw \quad . \quad . \quad . \quad (71)$$

B being as shown in Fig. 103.

Parallel slice dams may be employed up to 150 ft. high, but are usually confined to dams up to 100 ft. In any case the height is limited by two important considerations. A dam in true parallel slices cannot be built above a certain limited height without side buttresses. In the mass dam as shown in Fig. 104, where a=angle between W and R and ϕ the angle of the face at the toe with the vertical, assuming the masonry to weigh 140 lbs. per foot cube, the value of $p=7{\cdot}85$ tons per square foot, while

$$p\sec^2 a=10{\cdot}92 \text{ and } p\sec^2\phi=17{\cdot}90.$$

In the parallel slice dam the concentration of water loads in these separate slices increases a and ϕ so much that the maximum stresses are high. Assuming, for instance, a

parallel slice dam of the form shown in Figs. 105 and 106, the maximum stress will be 9·2 tons per square foot. P= 4·72 tons per square foot.

$$P \sec^2 \alpha = 9{\cdot}10 \quad . \quad . \quad . \quad (72)$$

$$P \sec^2 \phi = 25 \text{ to } 39 \quad . \quad . \quad . \quad (73)$$

consequently this latter consideration limits the height.

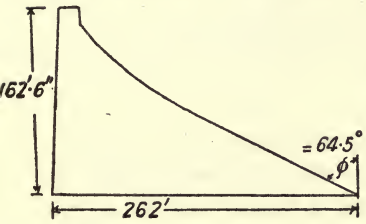


Fig. 104.

Fig. 105.

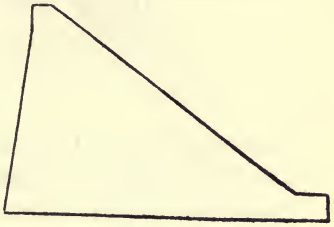


Fig. 106.

The dam is assumed to be one-third solid and two-thirds gap. That is, the buttresses are simply gravity buttresses.

Then there is the *Garrett* dam, of the form shown in Fig. 107, called after the originator of this design,

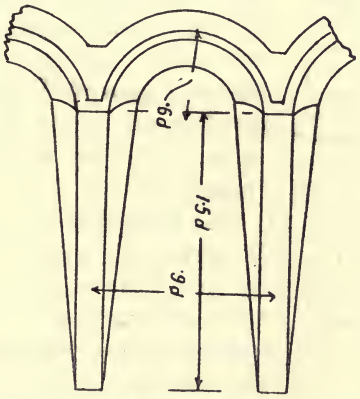


Fig. 107.

Captain Garrett, R.E. It may be used with economy for small dams.

We now come to an important type of dam lately introduced by Mr. Reginald Ryves, A.M.I.C.E.,* *vide* Figs. 108 and 109,

* Further information by the same expert may be had in *The Engineer*, February 24th, 1911 and March 3rd, 1911, and *Madras Government Paper*, P.W.D. (Irrigation.)

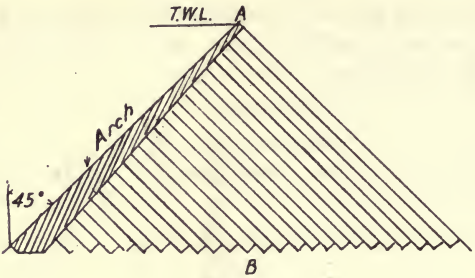


Fig. 108.

described by the designer in a paper read before the Concrete Institute on March 14th, 1912, and of which the following is an abstract. The best slope for the water face is, under normal conditions, 45°, and the dam then consists of inclined arches, of increasing thickness with increasing depths sloping at 45°, the abutments resting against the up-stream face of the buttresses, which are built up of layers all inclined at 45°, the thickness depending upon the material used, and the width of each layer being such that the thrust upon it is the maximum allowed thrust.

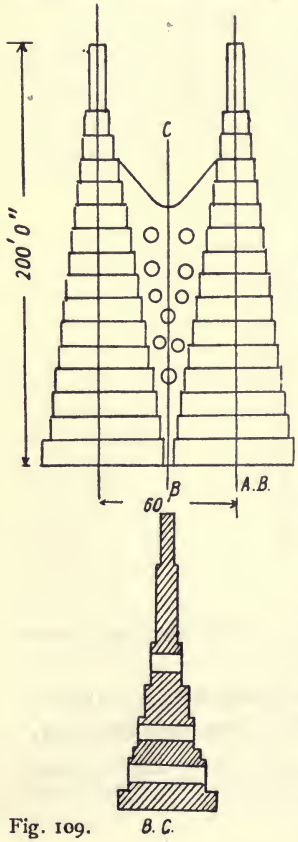


Fig. 109.

The calculation of thrust is made by adding the water load transmitted to any plane parallel to the face to the resolved part of the weight of masonry above the plane in question, the same method of calculation being adopted in every part of the buttress. The resolved part of the weight of the arch is, in each case, added to the water load. The water loads are thus transmitted by direct thrust, the layers being bonded together, but left free to take up the deformation under the water load by

sliding on one another. If the rock will not take the load at an angle of 45°, the water face may be made flatter and longer. The typical design is, however, that with a 45° slope.

This adoption of a general stress, instead of one varying from nearly zero to some maximum, is an important element of economy.

Comparing this dam with a mass dam, under certain conditions of site and costs of material, the following, for dams 162½ ft. high, was found :—

Mass dam, maximum pressure (Bouvier) 10 tons per square foot	Masonry, 100 units.
Thrust buttress dam, minimum arch thickness 6 ft., stress in arches 15 tons per square foot, in the buttresses 10 tons per square foot	95 units.
Thrust buttress dam with a minimum arch thickness of 4 ft.	92 units.

The difference would be considerably greater if we made, for the mass dam, $p \sec^2 \phi = 10$, or even $p \sec^2 \phi = 12$.

If the rock is not strong enough to take as high a maximum stress as that allowed in the buttresses, the layers can be footed out to a greater width below ground level. There is, for each stress allowed, a limiting height, which brings the buttresses together at their bases. With a stress of 10 tons per square foot the height is a little over 200 ft., and with a stress of 16 tons per square foot about 300 ft.

Such a dam may be built of materials such as slabs of limestone or hard shale, which are not suitable for the rubble masonry or mass concrete of a mass gravity dam.

The definite propositions which the author puts forward are :—

1. That there is no precedent for the building of a mass gravity dam in an unassisted length more than 150 ft. clear of the ground.

2. That our present ideas as to maximum stresses

allow of the building of such a dam for heights only slightly in excess of this.

3. That since the improbability of the stress actually reaching $p \sec^2 \phi$ increases as the slope at the down-stream toe is flattened, we may, for heights above 150 ft. and for angles flatter than 45°, allow one maximum computed stress for $p \sec^2 \alpha$, and a computed stress for $p \sec^2 \phi$ some 50 per cent higher.

4. That interrupted dams, the gaps being spanned by horizontal or inclined arches, or by beams, offer the important advantages :—

(*a*) Economy of material ;
(*b*) Absence of contraction cracks and temperature stresses ;
(*c*) Maintenance of waterway during construction ;
(*d*) Cheapness of sluices ;
(*e*) In some cases a wider choice of materials and of methods of building ;
(*f*) And that the interrupted dam, or dam of spans, may be considered as an alternative to the mass gravity dam for great heights, especially as regards the height clear of the ground.

5. That the author's design for a thrust buttress dam is suitable for heights up to 200 ft., with materials allowing of a compression stress of 10 tons per square foot and up to 300 ft., if a stress of about 16 tons per square foot be allowable.

6. That none of the dams at present built are thrust buttress dams, either as regards the method of building or in practical effect.

7. That, in order to avoid tension near the up-stream toe of a high mass dam, or of a gravity buttress in a dam of spans, it is desirable to make the connection with the rock somewhat as follows : The up-stream third of the base a flat surface with no bonding and no trench, the middle third bonded but not deeply trenched, the down-stream third effectively bonded with the rock cut in faces at right angles to the thrust of the masonry. When a high dam is

built with a large part of its bulk below the ground surface, such as the Owton dam, New York water supply (*vide* Fig. 110), that part of the up-stream face may be relieved of water pressure by draining it directly through the dam.

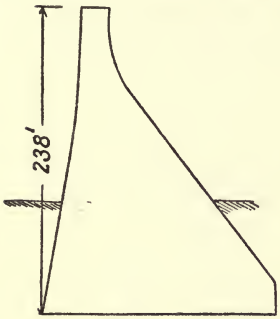


Fig. 110.

Practical Examples.

The Loch Arklet dam of the Glasgow Waterworks Extension, before referred to, is an example of modern practice and is illustrated in Fig. 111. A full description of the works is given in *The Engineer* of November 18th, 1910. The dam is not of solid masonry but of concrete faced with masonry, which is a favourite form. It is 1100 ft. long and faced with freestone ashlar. The top width is 12 ft. throughout, and the ground level width at centre 34 ft. A spillway 200 ft. long is provided for the overflow, and there are two culverts. The trench for the toe is sunk 10 ft. into solid rock, but the whole base of 34 ft. is also cut down to the rock.

The Vyrnwy dam in Wales, of the Liverpool Waterworks, was one of the first dams of masonry of any size constructed in Britain. It is built of solid masonry of the section shown in Fig. 112. The top of the dam forms the overflow weir or spillway, and above this a roadway is carried upon arches. The base of the dam rests on a solid rock bottom,

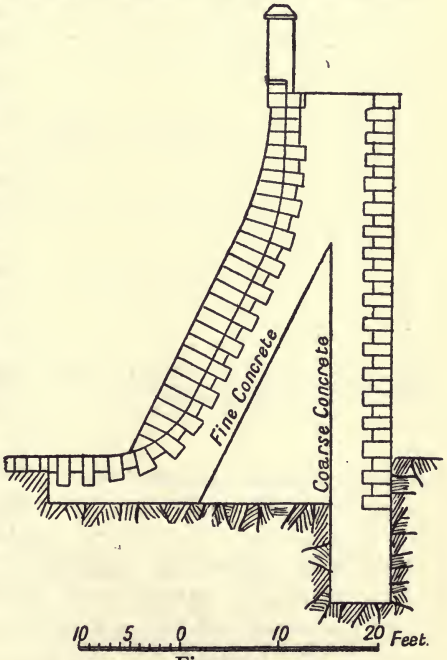


Fig. 111.

and the maximum stress on the dam is reported by Mr. G. F. Deacon as being 6·6 tons per square foot (vertical component). The Periyar dam (Fig. 113), Madras, India, is a concrete dam pure and simple. It was designed for a maximum stress intensity of 8 tons per square inch on a horizontal plane.* The Tytam dam of the Hong Kong Waterworks, at the instance of Sir Robert Rawlinson,† and shown in Figs. 114 and 115, is an example of a concrete and ashlar-faced dam of stepped profile and designed so that it can at some future date be extended in height as shown. The Abbeystead dam of the Lancaster Waterworks† was designed by Mr. James Mansergh, and affords the earliest‡ example of this class of work in Great Britain.

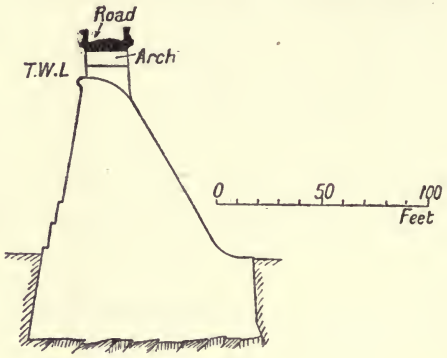


Fig. 112.

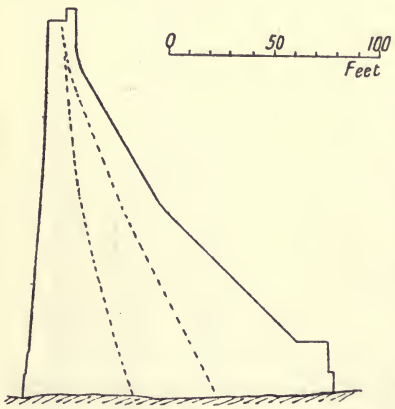


Fig. 113.

The top width of any dam is governed by local conditions. The triangular section with a thin edge is inadmissible in practice although sound in theory. It is usual to form a roadway over the top of the dam, carried on arches over the spillway. The spillway generally consists of a gentle curve (Rankine recommends the parabolic curve) and should be built very solid,

* *Engineering*, Vol. LIV, p. 654. 1893.
† *Trans. I.C.E.*, Vol. LXVIII.
‡ Tudsbery and Brightmore on Waterworks. Spon.

of hammer-dressed masonry, with a view to resisting the erosion of the water. It must be of such a length that the *maximum* flood will not rise above the crest more than 2 ft. Whilst the top of a masonry dam may, in whole or part, be made as a spillway,

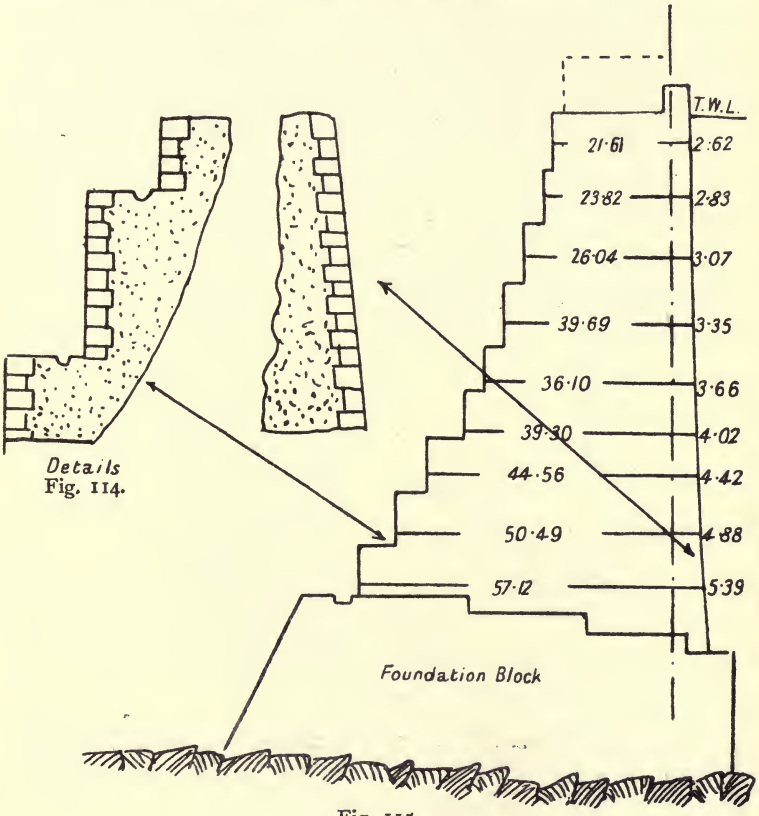


Fig. 115.

this may be quite a separate work, and is, in fact, in the case of earth dams. When a separate work a curved plan seems desirable. Such a form of spillway was adopted on the Panama Canal, and it is shown in Fig. 116.

It may be stated that those problems which relate to masonry dams take into consideration that the structure as a whole is practically rigid and not liable to deformation.

The calculations are simply to arrive at an accurate determination of the section of the dam under certain conditions. Whether, as is sometimes assumed, the plasticity of the materials of which dams are constructed leads to a yielding of the outer portions of their bases, and a consequent occurrence of maximum stress at some point considerably removed therefrom, or, as may be imagined, the outer portions sustain by an arch-like action the bulk of the superincumbent mass and the resulting pressure diminishes towards the middle—extended experimental knowledge of

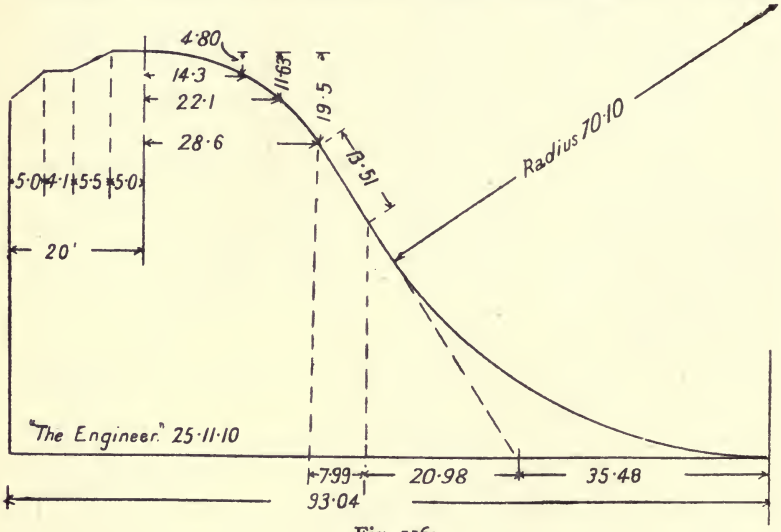


Fig. 116.

the internal stresses in masonry alone can form the basis of new laws and lead to advances in practice affecting the design of these structures.

Spillways.

In the case of earthen banks the spillway is an important masonry structure. It may take the form, as stated, of a stone weir, which in itself would be a small masonry dam and constructed on precisely the same principles, usually at one side of the bank and carried down to the rock on the valley side. Modern practice, however, is tending to the circular shaft as being more stable and giving a greater

waterway for a given quantity of masonry. This type has recently been adopted by the late Mr. O'Sullivan, City Engineer of Dublin, for the Roundwood Reservoir Extension, under the advice of Mr. Mansergh, and was adopted at the Middlesbrough (Blacton) Reservoir and elsewhere. The shaft is built on the inner side of the bank and delivers into a culvert, either carried through the bank or round the tail of it, the latter being preferable. The general profile of such an overflow basin is given in Fig. 117. A pit should be formed at the base to receive the impact of the water, which should under normal circumstances flow in a thin stream down the sides of the funnel rather than in the form of a cascade. In the case of dams of masonry the valve tower will often form part of the dam, as in the Tytam dam of the Hong Kong works. The well (there may be more than one) should contain the draw-off pipes and valves at different levels according to waterworks practice, led through the dam in a dry culvert, and may or may not contain a scour sluice. (The subject of culverts will be treated in the next chapter.) The top of the valve well is usually in the form of a masonry tower, which is generally designed with architectural features (e.g. Howden Dam, Derwent Valley Works). In the case of earth dams, and in some cases masonry dams, an independent valve tower is provided. This may be of cast iron (as described in the author's *Civil Engineering Practice*, p. 560) or of masonry. In the case of a masonry tower the wall becomes a cylinder, acted on externally by the water pressure, but the thickness of masonry necessary to ensure a solid structure is generally greatly in excess of what would be required to resist such pressure, and it is in consequence not usually considered. A masonry valve tower is generally designed

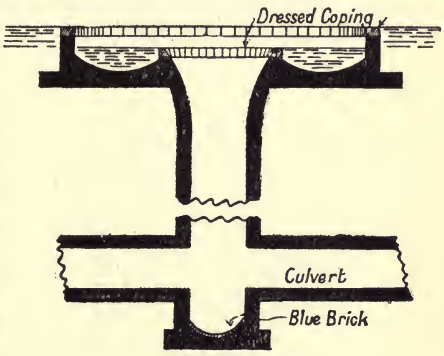


Fig. 117.

with a view to artistic appearance. Cast-iron towers, built in sections, are now almost invariably adopted for this purpose, being cheaper and more easily erected. For further information on masonry dams, see: *Annales des Ponts et Chaussées*, 1872 and 1897. *Comptes Rendus de l'Académie des Sciences*, 1895. *Drapers Co. Research Memoir*, London, 1904. *Engineering*, May 12th, 1905; and *The Engineer*, January 5th, 1872. *Trans. I.C.E.*, Vol. CLXXII. *Trans. American Soc. of Civil Engineers*, XIX. Wegmann, *Masonry Dams*, 4th Ed., New York. "Vyrnwy Dam," Report by Major-Gen. Sir Andrew Clarke, Liverpool, 1866. *Masonry Dams*, by Courtenay (Crosby, Lockwood). *Waterworks*, by Burton and Humber, and the very full details of the New York Waterworks, lately published by the Commissioners for this work.

CHAPTER VI

MASONRY BRIDGES

THE subject of masonry bridges is still an important one to the engineer, despite the fact that steel has done a great deal to supersede the mason and his ancient craft. In a masonry bridge there are three main parts, the arch which supports the load, the abutments which take the thrust of the arch, and the wing walls. There are also the spandrels, parapets, and foundations, which all demand attention. In order to understand the sequel Fig. 118 should be studied, which gives the various definitions of the terms used in speaking of the arch. Having decided on the form of bridge the first thing to do is to calculate the thrust on the arch. This may be done graphically and by formulæ. The keystone may be decided upon by a formula which is merely empirical and gives a depth really in excess of actual requirements, although it will be seen from the following list that it has usually been exceeded.

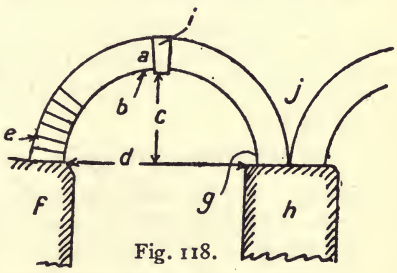


Fig. 118.

a=crown *b*=intrados *c*=rise
d=span *e*=extrados *f*=abutment
g=haunch *h*=pier *i*=keystone
j=spandrel

If R=radius of arch at the crown in feet, then for a single arch the depth of keystone in feet

$$=\sqrt{\cdot 12 \text{ Radius}} \quad . \quad . \quad . \quad (74)$$

but if the arch is one of a series then this value is

$$\sqrt{\cdot 17 \text{ Radius}} \quad . \quad . \quad . \quad (75)$$

The following is a list of what has been adopted in some important works by eminent engineers :—

(*a*) Grosvenor Bridge at Chester, designed by Hartley and Harrison and carried out by Trubshaw, being a segmental arch, 200 ft. span, 42 ft. rise, with 140 ft. radius, keystone is 4 ft. deep, ·1 ft. less than required by formula.

(*b*) Bridge over Severn at Gloucester, designed by Telford, an elliptical arch with span of 150 ft. and rise of 35 ft., and radius at crown of 160·7 ft., keystone 4ft. 6 ins. deep or ·11 more than required.

(*c*) Maidenhead Bridge, Great Western Railway, designed by I. K. Brunel, elliptical, span 128 ft., rise 24 ft. 3 ins., radius at crown 169 ft., and depth of keystone 5 ft. 3 ins. or ·11 less than was calculated. This bridge is considered a very bold piece of work (characteristic of the engineer), the actual arch being of brick in cement.

(*d*) London Bridge, a series of elliptical arches, with a span of 152 ft., and a radius at crown of 162 ft., depth of keystone 5 ft. or 3 ins. under calculated depth (engineer : Rennie).

(*e*) Waterloo Bridge, a series of elliptical arches, with a span of 120 ft., and rise 32 ft., and radius at the crown 112 ft. 6 ins., depth of keystone 5 ft. or $7\frac{1}{2}$ ins. in excess of that required.

(*f*) Blackfriars Bridge, nine elliptical arches of 100 ft. span, depth of keystone 6·58 ft. (engineer : Mylne).

(*g*) Pontyprydd Bridge over Taff, S. Wales, 140 ft. span, one circular arch, depth of keystone 2 ft. 6 ins.

It would be false economy and imprudence to radically depart from the above formulæ because the depth of any arch must be such as will prevent any likely deviation of the line of pressures, which might take place under partial loading, such as would be the case with a steam-roller or locomotive stopping half-way over the bridge. The loads on a large arch are less as compared with the weight of the arch than in a small arch, and in consequence the depth of keystone is made to increase in proportion to the square root of the radius at the crown.

Now if R=radius of curvature at the crown, and d= depth of keystone, and b=breadth of arch all in feet, let W=vertical weight on every square foot of keystone, including its own weight, and P=horizontal pressure on the keystone, and c the crushing strength of stone composing the arch per square foot in lbs., then

$$P=RbW \quad . \quad . \quad . \quad . \quad (76)$$

and the stability of the arch against crushing is proportional to

$$F=\frac{dc}{RW} \quad . \quad . \quad . \quad . \quad (77)$$

which is the factor of safety of the arch referred to the crushing strength of the keystone.

In those bridges before tabulated the value of F= Maidenhead 3, Grosvenor Bridge 22, London Bridge 40, Waterloo Bridge 68, which proves the statement that the Maidenhead Bridge was a very bold venture indeed.

The General Theory of Arches

The following paragraphs are, for the most part, taken from the author's *Civil Engineering Practice*:—

In linear arches the intrados is the actual line of that arch, the extrados being a line above joining the ordinates, and of a length proportional to the load. It is obvious that the extrados of a catenary or transformed catenary is a horizontal line. Now the thrust round any circular ring under a uniform pressure per square foot of circumference ×radius in feet is denoted by

$$pr \quad . \quad . \quad . \quad . \quad (78)$$

but from such a circle we can project an ellipse on which the vertical ordinates remain the same, but the horizontal increase (or decrease) in a certain ratio, c. Then, as above, pr still holds good for the vertical pressures, but the horizontal ones now become

$$cpr \quad . \quad . \quad . \quad . \quad (79)$$

the total thrusts being as the semidiameters or axes to which they are parallel. But the intensities of the pressures

are as the axes squared, or, conversely, the ratio of the axes of the rib are as $\sqrt{\text{of}}$ the ratio of intensities of principal pressures; hence c, as mentioned above in Formula 79, is equal to

$$c=\sqrt{\frac{P_y}{P_x}} \quad . \quad . \quad . \quad . \quad (80)$$

P_x and P_y being the intensities of the vertical and horizontal pressures respectively. In any linear arch the radius must be inversely as the pressure. The centre of resistance of every arch must be in the middle third of each arch stone, so as none of the joints tend to open inside or outside. This line approaches (in most cases) the linear arch referred to above, and must be balanced by a pressure from without, that is to say, P_y must not be negative (*ante*). If P_y does happen to be negative, then we have to introduce into the construction forms of ties as hoop iron or iron cramps.* But the linear arch, instead of being the line of resistance of the arch, often becomes the intrados.

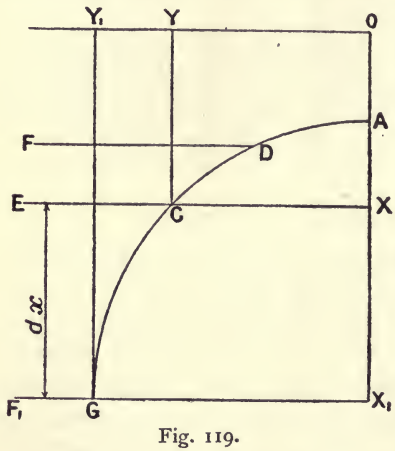


Fig. 119.

Now consider any linear arch A G, as shown in Figs. 119 and 120, and draw O C parallel to a tangent at C, and set *ob* vertical, make *oh*=load on *ac* (=pS as stated before), S being the length of the arc, then *hc* being drawn horizontal to cut *oc* gives us the horizontal component of the thrust along the rib at C. This construction, however, fails at the crown, and we resort to the formula (as in Formula 78)

$$T=pr \quad . \quad . \quad . \quad . \quad (81)$$

In the same way we draw oh_1h_1g and any convenient

* This form of construction is adopted in ferro-concrete.

number of such points so found. Then $hc-h_1g$ gives us the thrust on C E, F_1G. It will thus be seen that H_1 is less than H. Therefore if this arch is to be in equilibrium, pressure is required through those layers in which H increases in going downwards. This stress may be either a tension from without or a pressure from within. Now Kd=maximum horizontal thrust. Now such a point *d*, Fig. 120, where the thrust on the rib is parallel to *od*, is the point of max. horizontal thrust; this point is known as the point of rupture of the arch, and the angle *doa* (i_0)=angle of rupture. The above reasoning leads us to infer that the catenary is one of those few curves in which the length of the arc between any two points can be expressed in terms of the position of those points, and the area bounded by such an arc expressed by a simple formula; also if a horizontal line be drawn at any particular distance below the lowest point of the catenary, the height of any point on the curve above that line=the tension in the hanging chain at the point chosen. That particular distance is found by a formula which is too complex to give here.

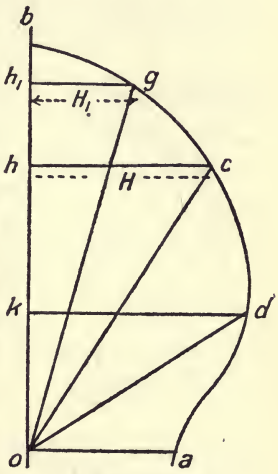


Fig. 120.

This horizontal line is known as the directrix. Another important formula in reference to arches is that to find the conjugate pressure P_y (referred to in Formula 80) exerted in any horizontal layer. Referring to Fig. 120, making $hc-h_1c_1=H$, then

$$P_y=\pm\frac{H}{dx} \quad . \quad . \quad . \quad . \quad (82)$$

and it will be negative under conditions explained when referring to tension from without or pressure from within being required.

The first step to ascertain if any arch is stable is to assume a linear arch parallel to the soffit, and by Formula 82 a series of values for P_y may be found in all cases positive; if

so, the linear arch drawn through the middle third of every arch ring will be the line of pressures. If P_y has some negative values, pressure from without is required. So up to this point where P_y becomes negative the backing has to be built up with square side joints, because in the cases where there is a negative P_y there will be a point of rupture, and it is below this point that such pressure has to be applied by means of this solid backing.

In most circular arches, in practice, this angle of rupture is between 45° and 55° to the horizontal, so it is usual in practice to carry up the solid backing to where the arch is so inclined at 45° to 55°, as at C in Fig. 121, also it is very near accuracy to say that the horizontal thrust in any arch =weight supported between the crown and part of the soffit at 45°=R F C V U in Fig. 121, and also in that figure the line Q F S is the line up to which the backing should in practice be built before striking the centres, P Q being equal to the mean square of the thickness of the arch and the radius at the intrados

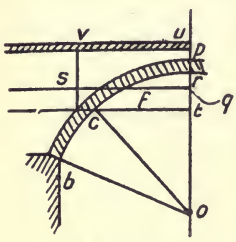


Fig. 121.

$$= \sqrt{PA \times AO} \quad . \quad . \quad . \quad (83)$$

The above reasoning applies to arches equal to or greater than the quadrant of a circle. If less than a quadrant we get the horizontal thrust by taking the weight of half the arch with its load and multiplying by the cotangent of its inclination to the horizontal at the springing. The part of the abutments next below the bed joints at springing must be able to resist the whole thrust of the arch as calculated above, being equal (with fair accuracy) to weight supported between the crown and part of the soffit at 45°. To this must be added the weight of half the arch; and if there is a thrust on the spandrel, viz. if H_1 is greater than H, *vide* Fig. 120 (in which figure, however, it is less than H), the part above the springing has to resist it. Large abutments and piers usually have archways in themselves, which rest upon inverted arches. By the use of such we can safely reduce the cubic capacity of the piers or

abutment by one-third. The piers are usually made one-fifth to one-seventh the radius at the crown in thickness. Each pier must have sufficient stability in itself to resist a thrust equal to the travelling load per foot-run (in case of railways it is reckoned at 2 tons per foot-run) × radius at crown in feet, and this accounts for the usual practice of having piers equal one-seventh span in thickness suitable.

Stress.

In any polygonal frame whose several parts are in equilibrio, the horizontal stress or strain at all the joints is the same, and the stresses in the several parts can be represented by lines drawn parallel to those parts, and limited by a given vertical line as in Fig. 122. Then the parts which the vertical line is cut into represent the vertical loads at the points in question. If all these parts are in equilibrio, the radiating lines will converge at a common point, as in Fig. 122. This figure treats a polygonal frame Y Z, being divided to scale to represent the weights of each division. Then from each of the points Y L M N Z lines are drawn parallel to the members. These are the radiating lines just mentioned, meeting at a common point X. The lines X Y, etc., represent the stress in the parts to which they are respectively parallel; and if X P is drawn perpendicular to Y Z, it will represent the horizontal component of the load at each of the points in question, which means to say that such a quantity is the same in each member.

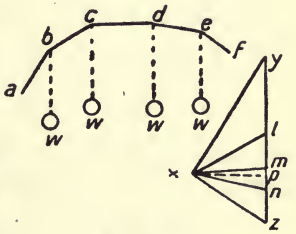


Fig. 122.

Load.

Each stone in any arch is subject to three different pressures, viz. :—

(1) Its own weight,

(2) The vertical load above it, and

(3) The side pressure from each of the two contiguous stones; and the resultant of all three is in the line of resistance of that arch. Now, for equilibrium, the vertical

depth of an arch at any point must be inversely as the radius at that point, and directly proportional to the cube of a line parallel to a tangent at the point in question, Fig. 123, that is to say, at right angles to the radius at that point. Then, in reference to Fig. 123, the following reasoning will hold good, viz. :—

$$RM : KL :: \frac{PB^3}{BL} : \frac{PQ^3}{BM} \quad . \quad . \quad (84)$$

P B being parallel to a tangent at R and P Q to a tangent at K. It will be seen that the arch in question will not be in equilibrium. Now for a circular arch the conditions of

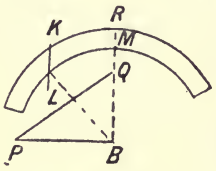


Fig. 123.

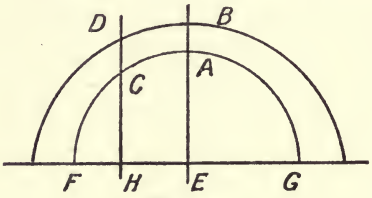


Fig. 124.

equilibrium are as in Fig. 124, when, having determined depth of keystone, we can make any depth D C

$$= DC = BA \times \frac{AE^3}{CH^3} \quad . \quad . \quad . \quad (85)$$

But if the arch is elliptical, as in Fig. 125, D C is equal to

$$BA \times \frac{AE^3}{CH^3} \quad . \quad . \quad . \quad (86)$$

But in this case A E=½ the minor axis and C H=the vertical depth from C to a horizontal line passing through the centre of the major axis. When such conditions are carried out the arch is in equilibrium.

Failure.

Arches may fail by the sliding of the voussoirs (or arch stones), which is not a likely occurrence, unless the line of pressure makes an angle equal to or greater than the limiting angle of resistance with a perpendicular to the joint; and as the limiting angle of resistance in such work is

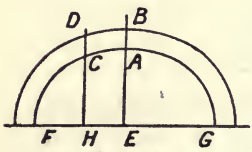


Fig. 125.

usually 30°, we are not likely to see such failure. But, on the other hand, an arch may quite easily fail by turning on some of the joints, as in Figs. 126 and 127. When the line of pressure lies outside the arch, as in Fig. 126, the inner joints tend to open; if, on the other hand, it lies inside the arch, the outer joints tend to open. Hence the very necessary use in arch design of Formulæ 84, 85, and 86. Such a failure as in Fig. 126 would be due to too great a load, while that in Fig. 127 is due to insufficient depth at

Fig. 126.

Fig. 127.

the crown. Due to conditions below stated, an arch has the same pressure tending to crush the keystone as it has on its abutments, and the pressure=load per square foot on the keystone × radius in feet at that keystone.

Elliptical Arches.

The elliptical arch is set out by means of a trammel, and if such a trammel is not handy, one may be constructed of paper, as shown in Fig. 128. First set A B=span and C D=rise, then by sliding *c* down the centre line and *b*

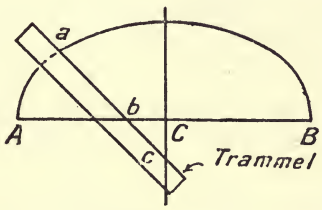


Fig. 128.

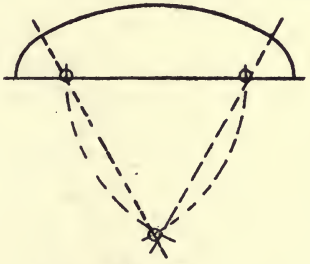


Fig. 129.

along the springing line the curve of the ellipse is traced out. An elliptical arch may also be drawn with three centres, two having a radius=$\frac{1}{6}$ span and the other $\frac{5}{6}$, as shown in Fig. 129. An ellipse may be set out as shown in Figs. 130 and 131.

If a full-sized curve be required for setting out the arch,

and the bricks rubbed or the stone voussoirs dressed to fit, a geometrical method is preferable, so that the joints can all be drawn normal to the curve, as in Fig. 130, then in order to set off the stone voussoirs so that they may increase in size towards the abutments, describe a semicircle with any radius from the intersection of the arcs. Divide this semicircle equally into the same number of parts as the number of voussoirs required, and from the centre point draw lines through these divisions to cut the ellipse. Then to obtain the direction of the joint normal to the curve, join any of these points as A to the foci F_1 and F_2. Bisect the angle F_1AF_2 and produce the bisection line, which will give a normal to the curve. If an elliptical curve has to be set out on the ground, as for a wing wall, the method of

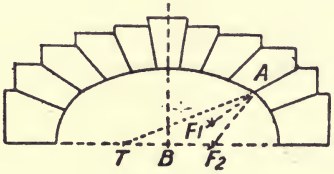


Fig. 130.

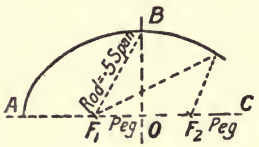


Fig. 131.

two pegs and a string is best, as in Fig. 131. Let A C be the major axis and O B half the minor axis. With centre B and radius equal to A O or O C describe arcs cutting A C in F_1 and F_2. These will be the foci of the ellipse. Then cut the string to a length sufficient to encircle the points F_1, F_2, and C, or F_2, F_1, and A. A marker should then be placed in the loop of the string and moved round, at the same time keeping the string perfectly taut, and an elliptical curve will be produced.

The stresses in an arch can be found by graphic construction, the following abstract on the subject being from the author's *Manual of Civil Engineering Practice*. Figs. 132 to 134 must be referred to.

The elevation of arch is shown in Fig. 132, together with the loading, and it will be seen that a load of 2 tons over the haunches has been taken into account for the weight of traction engines, etc., upon 1 ft. width across bridge. Set down the load line 1 to 13 in Fig. 133, and on the right

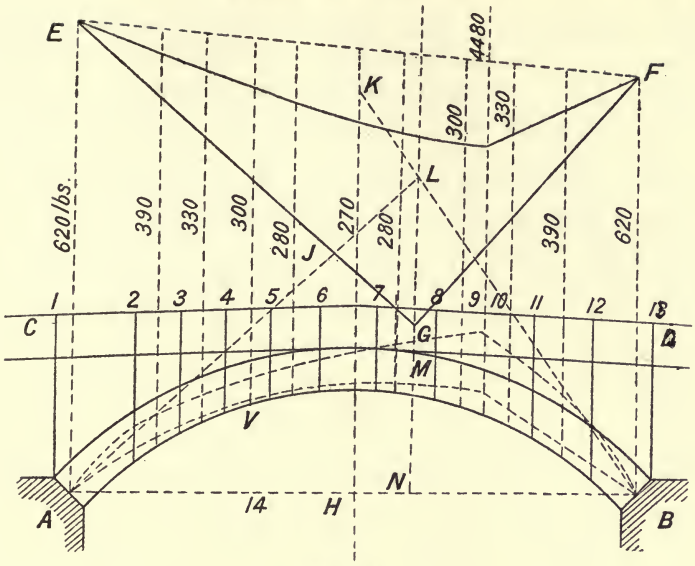


Fig. 132.

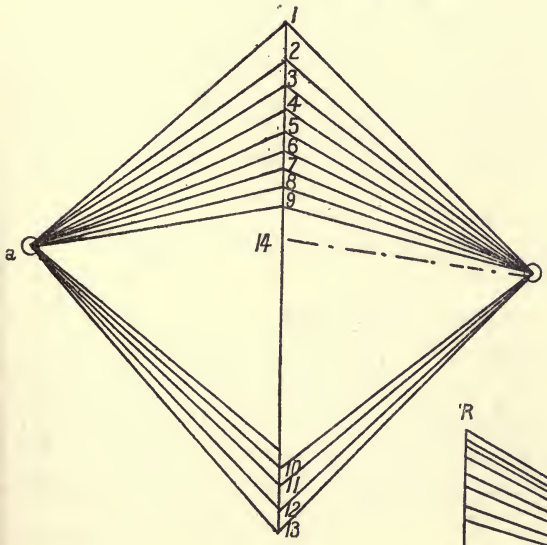


Fig. 133.

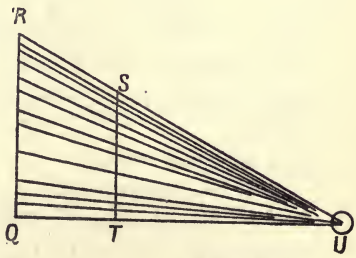


Fig. 134.

select any pole O; draw vectors, and parallel with these across their respective spaces construct the funicular polygon E F G; then G will give the position of the mean centre of gravity line. From O draw o to 14 parallel to line E F, giving the vertical reactions at the abutments. From A and B, the centres of the skewbacks, draw a horizontal line to meet a vertical line through the centre of arch in H. From H set up H J=1 to 14 and H K=14 to 13. Join A J and B K, and if produced they should intersect at L on the mean centre of gravity line. Then in Fig. 131 draw 1 to 14*a* parallel with A L and 13 to 14*a* parallel with B L. Draw the remaining vectors, and parallel to them draw in the curve of thrust A M B in Fig. 132. This is not the best position for the curve of thrust, and all the ordinates should be reduced proportionately. In Fig. 134 set up Q R=M N, Fig. 132, and at any horizontal distance Q T set up T S equal to the distance from N to the centre of the arch on the mean centre of gravity line. Join Q T and R S, and produce to meet in point U. Then the ordinates to curve A M B being set up on Q R and lines drawn to point U, the corresponding heights will be cut off on S T. These being set off in Fig. 132 will give the best position for the curve of thrust. The maximum stress will occur at V, and the calculations will be

$$\frac{W}{A}\pm\frac{M}{Z}=\frac{4550}{1\cdot33}\pm\frac{4550\times\frac{1}{2}}{\frac{1}{6}(1\times1\cdot33^2)}=3412\pm7678=11090 \text{ lbs.}$$

Abutments and Piers.

The abutments of bridges which withstand the thrust of an arch will be designed as retaining-walls; and knowing the direction of the resultant thrusts and the magnitude, the matter should not present any real difficulties. We have, however, a formula which will be of assistance in the design.

Let S=½ span,
,, V=versed line,
,, T=thickness of abutment at the springing.
All units are to be taken in feet.

Then

$$T = \cdot 17\left(\frac{S^2}{V} + V\right) \quad . \quad . \quad . \quad (87)$$

This applies to flat arches having a span greater than 150 ft.

For semicircular arches T may be taken as the thickness of abutment at a height above the springing $= \frac{2}{3}$ the radius.

As before stated, piers are usually made one-seventh span of arch in thickness, but in so doing, especially in high piers, it must be seen that the safe intensity of stress is not exceeded. For rubble stone 8 tons per square foot is ample, and 3 tons for good concrete ; soft sandstone would not be safe with more than $2\frac{1}{4}$. On rock foundations, however, we can safely have a load of 9 tons per square foot, and on ordinary earth $1\frac{1}{2}$ to 2. Foundations in sand should be at least 3 ft. deep and in clay 4 ft. It is very important for the engineer to rigidly inspect the site of the piers when the foundations are opened up. If the ground is not of a uniform character, unequal settlement will more than likely result. Broken rock, mud, soft clay, and running sand are always treacherous. All bad places should be cleaned out and filled up to a *level* surface with masonry or concrete. Undermining of piers must be specially guarded against ; induced scour will increase this tendency, and very thick piers will cause induced scour. Therefore in bad ground it is prudent to increase the sectional area by adding length to the piers where such a procedure would be deemed advisable. The chief difficulty in bridge work is the cofferdams for putting in foundations under water ; in soft ground piled foundations have to be used.

Fig. 135.

It was pointed out that there is a certain point in an arch to which the solid backing must be carried. If R= radius of the arch at the crown, and t=thickness, the distance X in Fig. 135=

$$Rt \quad . \quad . \quad . \quad . \quad (88)$$

and the level of the point of rupture y=

$$\frac{tR + \cdot 096R^2}{2t} \quad . \quad . \quad . \quad (89)$$

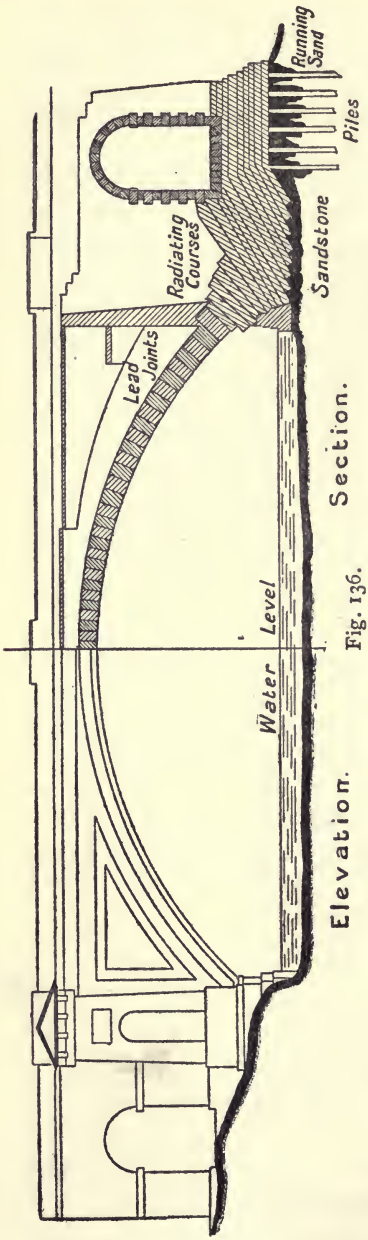


Fig. 136.

Examples of Masonry Bridges

No hard and fast rules can be laid down for the design of any masonry bridge. Each case has to be considered on its merits, and so long as the conditions before stated are complied with the form of arch, etc., may be left in the hands of the engineer.

Foremost among masonry bridges is the Grosvenor Bridge at Chester, which is illustrated in Fig. 136 in half section and half elevation. Some particulars of this bridge have been already given.* The appearance of the bridge as it stands to-day is very handsome and striking. The section is that of the north end. The clear waterway is 200 ft., the width between the parapets 33 ft. The arch is circular and of 200 ft. span and 42 ft. rise. The radius of curvature at the crown is 140 ft., the depth of the keystone 4 ft. The stone used was red sandstone,† plentiful in this district. The engineer was Mr. Hartley, of Liverpool. The

* Full details of the bridge are given in *Trans. I.C.E.*, Vol. I.

† The two first courses of the arch and face of abutments are granite, and three centre courses of arch and quoins of Anglesea limestone.

south abutment is founded on solid rock, and the foundations are of a simple nature. This rock terminates a few feet south of the north abutment and loose sand follows it. This was the reason for the type of foundation shown in the drawing, with piles. The radiating courses of the abutments are themselves interesting, as they are really part of the arch, and they are keyed into rock, which rock, in fact, forms the actual abutment. Lead joints were used in this bridge from the springing line to the point of rupture, as shown in the drawing, and under the actual springing course two wedges of lead were laid 1½ ins. thick on the face and tapering to nothing at the back. This was to guard against any undue settlement when the centres were struck (this will be referred to when the centering is considered) and to cause equal distribution of pressure over the surface of the joints due to the yielding nature of the lead. The experiment was quite successful. Lead was also used to set the keystones, strips of lead being placed on each side of the recess to receive them. The keystones, smeared with putty, were forced home by means of a small pile-driver.

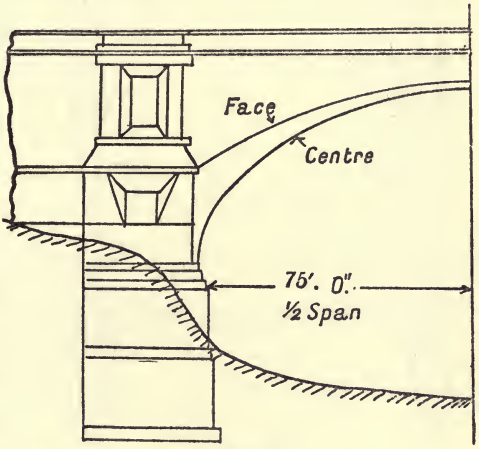


Fig. 137.

In Fig. 137 Telford's bridge over the Severn is illustrated, and in design is peculiar, inasmuch as the soffit of the arch is only straight at the centre cross-section. The result is that the curve of the intrados of the arch is flatter at the face than in the middle. The effect from an artistic point of view is good, but the real advantage is that floods will pass easier. There is also a saving of material, but the extra labour on centres and stonework more than counterbalance this.

Single-arch bridges are only economical for spans of about 150 ft., and above this intermediate piers must be built. Upon the sound foundation of such piers the whole stability of the structure will depend, and every precaution must be taken to ensure the stability of arches resting on piers by solid abutments. The abutments of any bridge are retaining-walls, whose thrust is equal to intensity to that found by Equation 87, and this thrust acts at right angles to the plane of the skewback. Upon this data, then, the abutment may be designed in the same way as any retaining-wall, as already pointed out. They are usually heavily counterforted at the back, the counterforts being in this case buttresses. In the case of river bridges the piers may be the most difficult part of the work. In shallow water a cofferdam of the form shown in Fig. 138 is all that is required, but in many cases iron cylinders will have to be sunk to a sound foundation. These cylinders are formed of cast-iron rings, as shown in Fig. 139, bolted together internally and lined with brickwork or concrete. During sinking the earth is excavated by means of mechanical excavators and, when sunk, they are filled with concrete. In many cases compressed air has to be used, the men working inside the cylinders. This system is much used, especially for the piers of steel bridges.

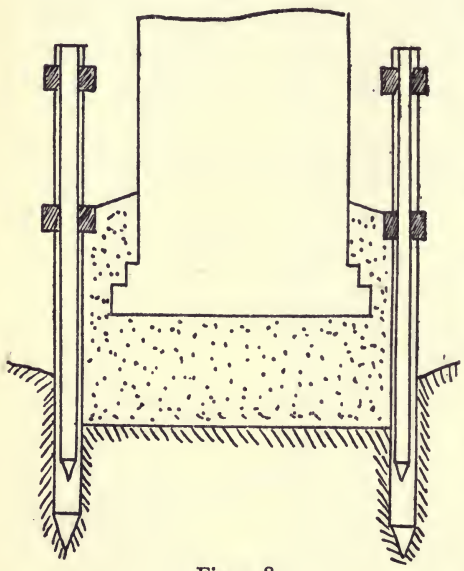


Fig. 138.

Cylinders of this kind were used on the Barrow and Suir bridges of the Cork and Rosslare section of the G.S.W. Railway (Ireland) by Sir Benjamin Baker, and for the piers

of the Chattaraviti Bridge of the Madras Railway (as shown in Fig. 139), by Mr. Hawkshaw, and subsequently filled with concrete. The cylinders, which are typical of modern practice, are described in a paper by Mr. E. W. Stoney,* of which the following is an abstract :—

" The cylinders have a diameter of 12 ft. throughout the lower portion, to within 9 ft. of the river-bed, at which point a conical tapering length is inserted, reducing the diameter to 9 ft. The 12-ft. portion was made in lengths of 6 ft., each length being cast in six segments with $1\frac{1}{4}$-in. thickness of metal, strengthened with internal flanges and feathers, and united by $1\frac{1}{4}$-in. bolts. The joints were caulked with iron-rust cement to within $\frac{3}{4}$ in. of the inner edge, this space being filled with a mixture of 1 part of sand to 1 of Portland cement to make the joints air-tight and fit for pneumatic work.

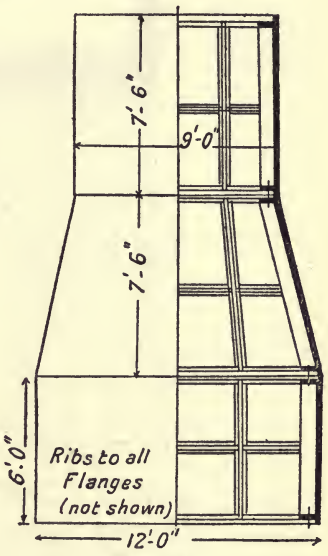


Fig. 139.

Each cylinder was furnished with a special bottom length of wrought iron, forming a cutting-ring 3 ft. in height built of $\frac{5}{8}$-in. plates. The cutting-rings, after being riveted together, were placed accurately in position on the dry river-bed, and were sunk 2 ft. 6 ins. by men digging inside them. Two lengths of cast-iron cylinder were then erected upon each cutting-ring, and when they had been bolted and caulked, a Priestman's crane and dredger, standing on the working lines of railway in the river-bed, continued excavating until the top had been sunk to the level of the ground. Two additional rings were then built up, forming a second length of 12 ft., and provision was now made for weighting the cylinders by placing across the top two pairs of hardwood beams 18 ft. by 18 ins. by 18 ins.,

* *Trans. I.C.E.*, Vol. CIII.

on which the 75-lb. rails were stacked. The load varied from 200 to 1120 rails, according to the depth of the cylinder and the nature of the strata it passed through. With large loads the height of the stack above the river-bed was often as much as 27 ft., so that the Priestman's dredger could not be used, and Wild's, Bell's, or Bull's dredgers were employed, being worked by steam-hoists by means of derrick poles."

The real duty of a bridge pier is to support the arch,

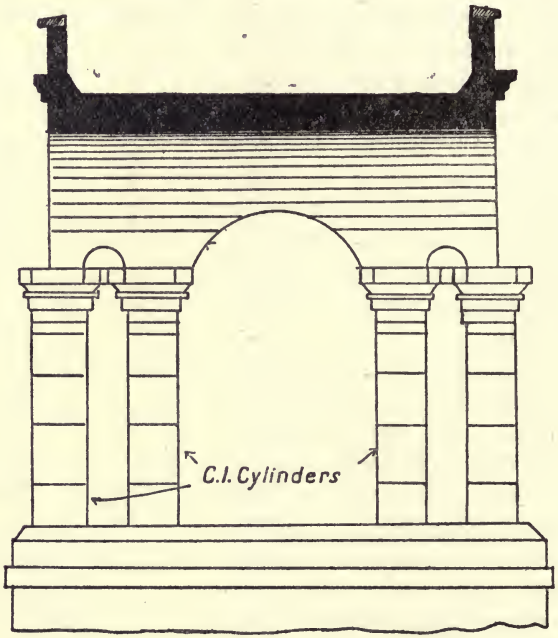
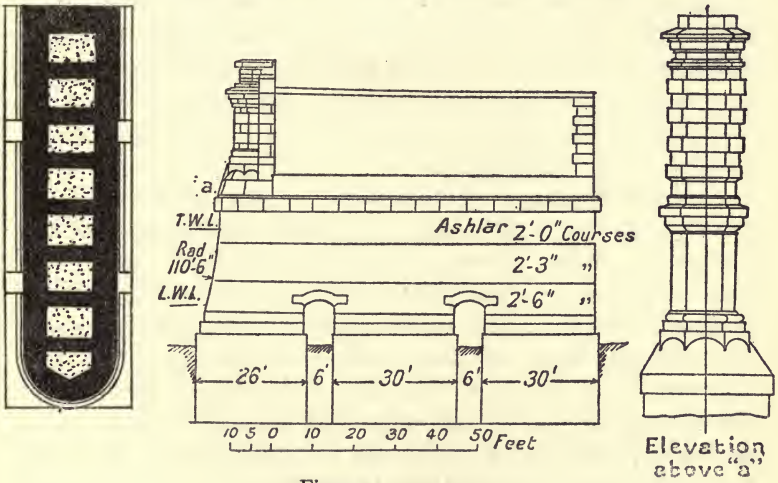


Fig. 140.

receive its weight, and transmit it to the foundation, and in consequence the lighter the pier the better, within safe limits. It should be able to withstand without injury the vertical pressure of the arch and its load, as well as that of the water and any accidental force (ice in cold climates) to which it might under extraordinary circumstances be liable to be exposed, and the base should be of such an area that the safe load on the subsoil or material forming the foundation is not exceeded. When these conditions are fulfilled all the

extra material is waste, and it has been the practice of engineers in the past to build masonry piers of sizes much in excess of that needed. The generally accepted rule is that they should be in thickness one-seventh the span of the arches, but, if built solid, this in many cases provides a lavish excess of material, and openings should be made from side to side with the top and bottom in the shape of small arches. The cylindrical pier as shown in Fig. 139 has much to recommend it for masonry bridges, although its adoption is rare in such cases. They could be continued



Figs. 141, 142, 143.

upwards in the form of a circular masonry shaft, across two or more of which the arch would rest, as shown in Fig. 140. The resulting piers are light, stable, and offer very little resistance to the free passage of water. Of course, in cases when simple cofferdams can be made and a firm foundation is to be had within a few feet of the even bed this and the following construction are quite unnecessary.

Piers sunk with caissons have been used on many important works. The piers of the Blackfriars Railway Bridge, shown in Figs. 141, 142, 143, were sunk in this way, but the superstructure of this bridge is of iron. The caissons used * are rectangular, ranging from 30 ft. × 32 ft. to 30 ft. × 26 ft.

* *Trans. I.C.E.*, Vol. CI.

They consist of a single skin of wrought-iron plate $\frac{1}{2}$ in. thick, for the bottom 7 ft., and diminishing to $\frac{5}{16}$ in. at the level of high-water. The lower portion, or permanent caisson, is, in each case, 21 ft. in height; and above this a temporary caisson was carried up to a further height of 28 ft., this portion being bolted to the permanent caisson, and removed after the building of the pier inside was sufficiently advanced.

The caissons were stiffened vertically at the corners by angle-iron, and on the sides by tee-irons spaced at intervals of about 7 ft., and were further strengthened by temporary horizontal stiffeners placed about 4 ft. apart. For the latter purpose wrought-iron girders and diagonal struts were at first employed; but, owing to the difficulty of removing them under the great pressure, it was afterwards found better to employ timbering.

The cutting-edge of the caisson was of rolled iron, stiffened by means of triangular gussets butting against the underside of the lowermost horizontal girder, and other gussets were placed above the girder to prevent its yielding upwards. The pressure on the lower gussets proving somewhat greater than on those above, it was found in some instances that the girder bent slightly upwards and allowed the cutting-edge to be forced inwards to some extent. This necessarily impeded the descent of the caisson, and, after the experience gained in sinking the first caisson, the gussets of all the remaining caissons were strengthened. The bottom girder and gussets were designed to be left in permanently.

The upper horizontal frames were stiffened vertically by temporary timber puncheons fitted between them.

At the top of the permanent caisson thickening strips were riveted to the skin plates to give them additional width, and on these rested the temporary caisson.

The bottom of the temporary caisson consisted of inverted channel-irons, containing flat strips of vulcanized india-rubber, and the joint was tightened up by means of bolts spaced about 2 ft. apart round the edges of the caisson.

The vertical and horizontal stiffeners of the temporary caissons were generally similar to those of the permanent caissons although of a lighter description.

The caissons were weighted with kentledge to facilitate the sinking, and under this load the cutting-edge penetrated at first below the excavation ; but as the sinking progressed and the friction increased, it became necessary to excavate down to the cutting-edge ; and in some cases it was with difficulty that the caissons were forced down to their full depth. This was especially so in the case of the caissons of pier No. 4, which had to be sunk through thick layers of septaria embedded in the London clay.

To aid the descent of the caissons of this pier, hydraulic presses were applied, the purchase being obtained against the undersides of inverted trussed beams laid across the caissons, and secured to the piles of the staging surrounding the pier. When the presses at each corner of one of these caissons were exerting a pressure of 30 tons each, or 120 tons together, the total load on the caisson amounted to 390 tons, the hydraulic pressure being aided by the weight of the caisson, including timbering, etc., which was estimated at 143 tons, and also by the weight of the kentledge, amounting to 127 tons.

No matter what form a bridge pier takes, it must, below water, offer the least possible resistance to the flow of the water. This is fulfilled by making the ends of the pier either semicircular in plan or in a form approaching a vee. The former has been found to offer as little obstruction to the flow of water as any other shape, and is, moreover, less liable to damage in floods by floating bodies such as trees and ice in cold countries. When the piers are built, no matter what form they take, the engineer should not be too anxious to build up the arches. They must be allowed time to sink and settle, and should be loaded for the purpose with old rails of a weight equal to that at least of the arch, superstructure, and maximum load. In this state they should be left until, by repeated and accurate observations with a level, no further settlement is taking place, and only then, and not till then, is it wise to start and

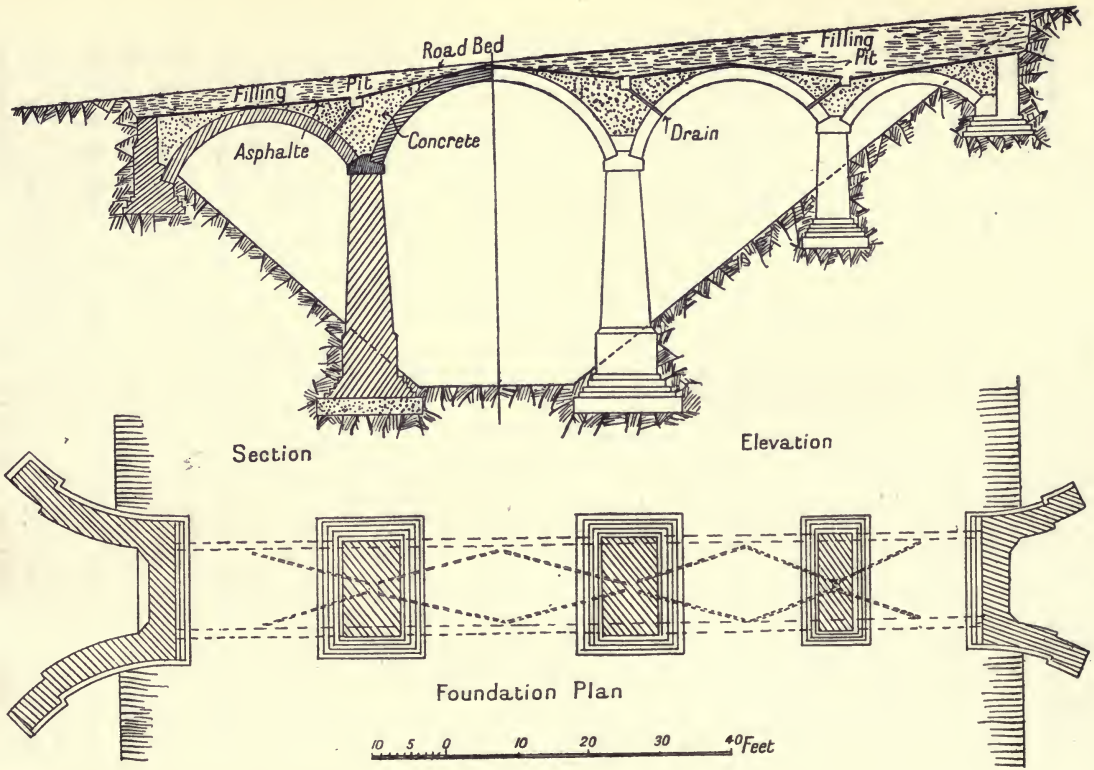

Road Bed
Filling
Pit
Filling
Pit
Asphalte
Concrete
Drain
Section
Elevation
Foundation Plan
10 5 0 10 20 30 40 Feet

Fig. 144.

work. In Fig. 144 is illustrated a road bridge over a deep railway cutting.

An example of a viaduct constructed over a flat estuary is that lately completed at Radcliffe over the River Trent for the Great Northern Railway Company by Mr. Alexander Ross. Some details are given in Figs. 144–5–6 and the following description is that given in *The Engineer* of December 30th, 1910 :—

" The first brick of the new viaduct was laid on January 19th, 1909, and the last portion opened for traffic on September 5th, 1910. The work was stopped for several weeks last winter by floods. The following is a description of the new viaduct and the method of erection : The total

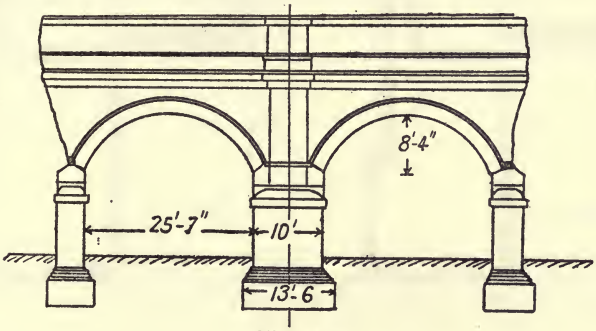


Fig. 145.

length of the viaduct, including the bridge over the River Trent, is 19 chs. The new portion consists of twenty-eight spans, eighteen of 24 ft. 11 ins. and ten of 25 ft. 7 ins. each, with a double abutment between them on the bank described above. The arches are divided into five sets, and are separated by stop piers 10 ft. in width, the intermediate piers being 4 ft. 1½ in. The arches are five rings in thickness, and are covered with blue brick paving. Cutwaters in brickwork have been formed at each end of the piers, as the floods from the river run in both directions from the bridge. Manholes have been formed on the stop piers, and large circular recesses on the central bank to form places for look-out men in times of flood.

" The foundations of the piers average 10 ft. in depth, going through a bed of gravel on to a bed of marl, which

forms part of the Keuper formation. The concrete in the foundations was made with six parts of broken slag, and gravel obtained during the progress of the work, and one of cement. The concrete was made of this strength as considerable difficulty was experienced at times with water in the foundations, which at the normal level of the river rose through the gravel to within an average of 6 ft. from the surface of the ground. The viaduct was built of local bricks faced with Staffordshire brindled bricks, the whole of the work being done in cement with mortar composed of

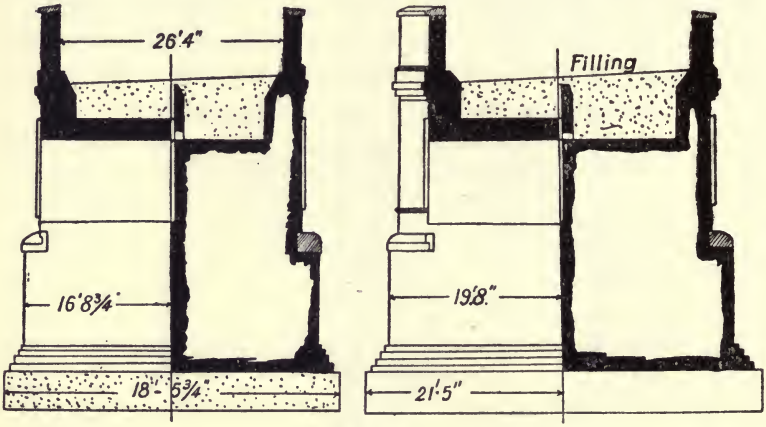


Fig. 146.

one part of cement to three of sand. The stone in the caps of the cutwaters and coping is Derbyshire.

"The following is a brief description of the method adopted in the carrying out of the work: In the first place, the three stop piers were built up to the springing level of the arches. A trestle was then fixed upon each of them up to the longitudes of the old viaduct, and was securely wedged to them, thus dividing the old structure into five parts, so as to minimize the risk during reconstruction. The work was then dealt with in sections between the stop piers. The intermediate piers were carried up to springing level and the centres fixed. Owing to these having to be built between the timbers of the existing viaduct, it was necessary to erect each one of them in pieces in place. The

laggings were then fixed for about 4 ft. on each side of the piers, and the piers raised to above springing level, about 4 ft. of the arch being turned on each side of the pier to enable this to be done. Trestles were then fixed on the top of each pier in a similar manner to that on the stop piers. The struts from the piles of the old viaduct were then taken out and temporary struts fixed in the opposite direction from the new brick piers to the longitudinals. This enabled the lagging to be put on over the rest of the centres, the only portion of the old viaduct coming through the lagging being the piles. The arches were then turned in the usual way, leaving the piles passing through them. So soon as the haunches were filled up and sufficiently set to bear the weight, the whole was paved with blue bricks; packings were then put under the longitudinals so as to bring the weight on to the arches. The tops of the piles were sawn off underneath and withdrawn from the brickwork, the holes they had occupied being filled up."

Road Bridges.

Small road bridges, such as would serve to carry a road over a canal, railway, or small stream, are often made in the form of a single masonry arch; a large number of such bridges were executed at the beginning of the nineteenth century, during the canal era, when steel was not used for the purpose, and Portland cement unknown. Hence we usually find them built of brick or stone, set in ordinary lime mortar. Many of these bridges are now in a fine state of preservation, but it is usual to find many of the approaches steep and tortuous, and with the arches springing from below the river-bed. The crowns usually raise objectionable humps on the road surface. They are usually too high, that is the height of the soffit of the arch gives no assistance to the passage of flood waters, while the spans are insufficient for this purpose. Again, in spite of the fact that such bridges often find place on swift mountain streams, no provision is made to prevent scour by means of any paving. The bridges are usually insufficiently strong to carry traction engines and steam-rollers.

The first consideration is design, that is, from an architectural point of view, while the clear span and height under the floor are matters of local consideration, and also as to the number of spans. Small bridges have rarely more than one span, and in any case piers have the disadvantage of impeding the flow. Hence we conclude that a flat-topped bridge is most suitable for roads having straight approaches on an easy gradient; 100 years ago, 10 ft. width was considered ample; this, however, will not allow of two vehicles passing on the bridge, which, on a main road with much fast motor traffic, is a disadvantage, and the width should be the *full width* of the roadway, together with one or two footpaths, according to circumstances. Over small mountain streams it is an advantage for the arch to spring from the river-banks, so as not to interfere with the flow, and the discharging capacity must be *at least* equal to the bridge above it, if there is one.

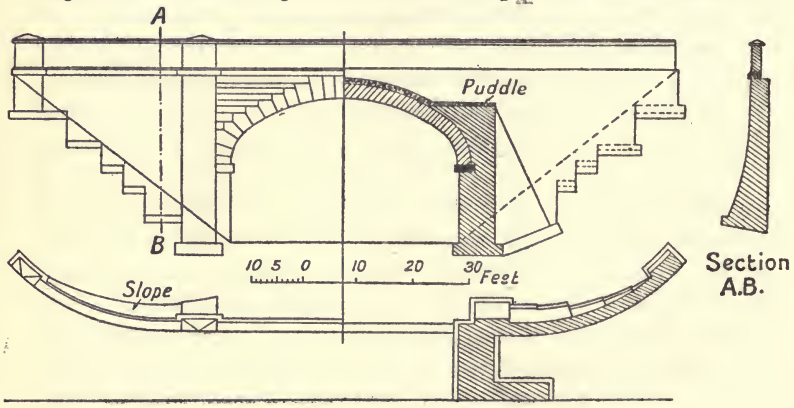
A skew bridge is to be avoided. At the same time it is a bad practice to place a bridge at right angles to a tortuous stream and the road, which more than likely runs parallel with the stream, turned abruptly to meet it. This is often unavoidable in crossing railways. In such cases, then, unless it would be policy to straighten the course of the river for some distance, a skew bridge will often prove the best, because any interference with the natural course of a river must be undertaken with caution, by reason of the fact that it will always tend to resume its former direction. The most important consideration of all is the foundations, the nature of which must be ascertained before any attempt can be made at the design. It is not unusual to find gravel in this direction, and the foundations should be carried down at least 5 ft. under the lowest level of the river-bed, unless hard rock is encountered. In the meantime concrete bags are very useful where water is troublesome, while on bog land (much of which is encountered in Ireland) a raft foundation of timber (old railway sleepers) planks laid at right angles to the face of the abutments are very useful, especially in such cases where there is about 18 ins. of hard earth above the bog, which it is unwise to

disturb. Trial pits are of great service in ascertaining the exact nature of the subsoil, but the money available for road bridges is usually not enough to warrant their use.

The piers and abutments of small bridges are not usually subject to much calculation, 3 to 4 ft. being the usual mean thickness, the face being vertical and the back stepped as pointed out under Retaining-Walls. If for arched bridges they must withstand the greatest thrust of the arch, and if for supporting girders, the greatest dead and live loads on the bridge, and also the thrust of the earth backing, due not alone to its actual weight, but to the horizontal component of the live load on the roadway immediately above it.

Concrete abutments are economical, especially if *displacers* are used. These should be of rough stone, and carefully set to break joint, and should be at least 6 ins. apart on plan. They should be laid flat, and not greater in size than 18 ins. × 12 ins. × 9 ins. The faces of concrete piers and abutments should, of course, always be finished in 1 to 1 cement. Piers, if high, sometimes have battering faces. The walls, which are built up from the arch to roadway level and termed spandrel walls, must be of sufficient strength to withstand the pressure of the filling and the horizontal component of the load on it in the same way as the abutments. It is also usual to introduce two or more walls between the two outer spandrel walls and parallel to them. They are known as *internal spandrel walls*, and they are generally covered with flagstones, and the space below left hollow, or small arches may be turned between them instead, to support the road metalling. When it is necessary to build these spandrel walls of any height, they are strengthened by cross-walls parallel to the abutments. And it must be remembered that where the arched construction is employed, the outer walls are subject to the outward thrust of these arches. Immediately above the keystone of the arch a string course is run along the entire length of the outer spandrels, which continue upwards above this to about 3 ft. 6 ins. above roadway level. These walls are known as the *parapet* walls, and they are usually 12 to 18 ins.

thick, and surmounted by a skew or half-round coping, the latter being preferred in concrete work. In the case of heavy banks to support the approaches, wing walls must be built to retain them. They are subject to the same calculations as retaining-walls, but usually have a sloping face. Squared rubble masonry, laid dry and pointed in lime mortar, is a favourite and inexpensive form of construction. The back next to the road they retain is usually vertical. In most cases it will be advisable to pave the space between the abutments, and the best construction for the purpose is that of heavy stones 18 ins. deep laid on edge, with the longest dimension up and down stream.



Figs. 147, 148, and 149.

They should be well bonded together, and form, when complete, a channel of circular cross-section. Care must be taken, however, that the ground below is not likely to be undermined by the current, and a stone apron on the down-stream side is an advantage in such cases, to reduce the velocity of the water between the abutments. Concrete paving, though easy to construct, is not to be advised, owing to its more or less rapid wear under such conditions.

There is no special form of design for such road bridges. Simplicity and small first cost is usually essential to such works, and Figs. 147, 148, and 149 serve to show a typical example. Curved wing walls, as will be shown later on, while adding to the appearance, do not add to the

efficiency of the structure, while they do add considerably to the expense.

Construction in General

The masonry used for arches is usually ashlar or block-in-course. The beds of the stones should be as nearly perpendicular to the thrust in the arch ring as possible, and the side joints perpendicular to the beds and the soffit.

The bed joints in square arches are plane, but, as will be shown later, in skew arches they are curved. The bed joints in any arch must be very thin, if after-settlement is to be avoided, and the stones very accurately cut to templet. The backing should be coursed rubble, but is often of concrete, to which there is no real objection, but in the case of stepped voussoirs, shown in Fig. 147, masonry in courses of the same depth as the steps is the best construction. The backing must be carried to the depths found by a previous equation. The upper surface of the backing is made impervious to water by either clay puddle, neat cement, or asphalte, the latter being the most modern method, and the water which may percolate to this surface drained away. In the case of brick arches purpose-made bricks of tapering width, according to the radius of the arch, are the correct ones to use, but in practice are rarely used; it is usual to build such arches on concentric rings of a depth=depth of 1 brick, viz. $4\frac{1}{2}$ ins. All the bricks are laid as headers to the face, and the connection of the concentric rings depends entirely on the mortar used, which should be 2 to 1 cement mortar. Hoop-iron laid along and round the arch between each ring will increase the stability.

Centering.—To build a masonry or brick arch a centre is essential, and the design of large centres is a study of construction in itself and one in which the engineer has a wide field for his ingenuity. They are usually made of timber. The duty of the centre is to support the voussoirs of the arch during construction until the keystone is inserted. The upper surface corresponds accurately with the intrados of the arch, so that the stones placed upon it retain their

proper position until the arch is completed. It must be constructed so rigidly that it will not alter in form under the weight of the arch, and for large arches a steel-framed centre is, in the writer's opinion, worth the extra expense, but has rarely, if ever, been adopted, although such centres are largely used in a small form for sewers and culverts.

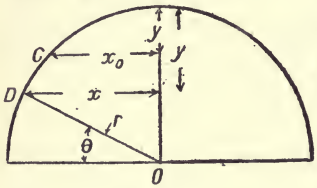


Fig. 150.

Taking now w=weight in lbs. per lineal foot of the intrados (or soffit) of any arch. Take any point D in that soffit and join that point with the centre O, Fig. 150. Note the angle that it makes with the horizontal and call it θ. Then the equation

$$P=w\cos\theta \quad . \quad . \quad . \quad . \quad . \quad (90)$$

will give us the *greatest* stress likely to come into the laggings (the narrow strips of wood attached to the ribs which actually support the voussoirs). Referring to Fig. 150 again, the normal pressure against the rib at the point D per lineal foot of the intrados, using the notation on the figure,

$$=P=w\cos\theta-\frac{1}{r}\int_{y0}^{y} wdy \quad . \quad . \quad . \quad (91)$$

r being the radius of curvation.

Again, if we want to find the vertical load between C and D due to the pressure of the arch stones, in order to calculate the scantling for a post, its value is

$$Q=\int_{x0}^{x} pdx \quad . \quad . \quad . \quad . \quad . \quad (92)$$

But we do not always support an arch centre on a series of posts—it is sometimes impossible. Then we have to treat it as a girder, and if s=the half span of such a girder, and we want to find the bending moment in the centre of the span,

$$M=Qs-\int_{x0}^{x1} pxdx \quad . \quad . \quad . \quad . \quad (93)$$

Again, on any segment of the rib immediately under the laggings, and inclined to the horizon θ' degrees, we have the total transverse load on that piece

$$=Z=Q\sec\theta' \quad . \quad . \quad . \quad . \quad (94)$$

The connection between the centering and the posts is made by means of *wedges* or *striking plates;* it is by loosening these that the arch is made to settle and take up its load. Regarding the period to do such preliminary loosening, the reader is referred to Arches. In the actual design of centerings there is nothing particularly abnormal. Experience and adaptability to circumstances are the principal factors. One of the most successful pieces of centering work (as well as the bridge itself, which we shall describe) was that of Mr. Hartley, a Liverpool engineer, for the Grosvenor Bridge at Chester. It is stated, however, that Mr. Hartley did *not*

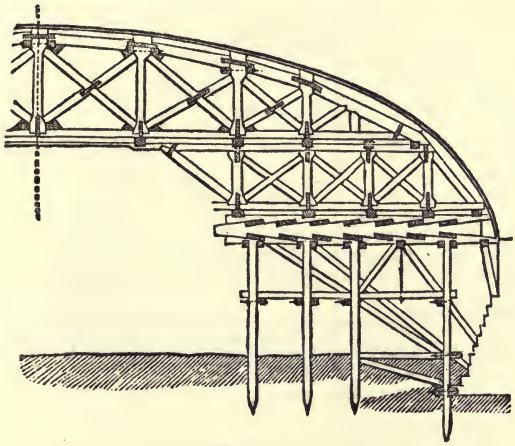


Fig. 151.

design the actual centre, but Mr. Trubshaw, the contractor. Four temporary piers had to be sunk in the river, on the top of each was a special cast-iron shoe, to receive the fan-like framings; these framings were six in number and strengthened by cross-pieces.* Another large centre is that designed for London Bridge and shown in Fig. 151, designed by Telford. Details of an ordinary piece of centering which would suit most cases of moderate span are given in Figs. 152–153, and for an elliptical arch in Fig. 154, and suitable for a bridge, the span of which is from 60 ft. to 100 ft. The struts that are marked A are birdsmouthed and fixed with dogs, and

* *Trans. I.C.E.*, Vol. I.

the struts that are marked B and the pieces marked C are bolted in pairs on the outsides. The pieces marked D are stretchers, which are carried across to the next rib, which should not be more than 9 ft. apart. The centre is raised by a continuous wedge, which is shown in Fig. 155.

A very heavy centre is shown in Fig. 156, while Figs. 157–8–9 give diagrams of suggestions for forms of centre. In the centering employed in the new building for the Woollen Merchants' Exchange in the Borough of Manhattan, N.Y., says a writer in *Building Age*, the opening is 14 ft.

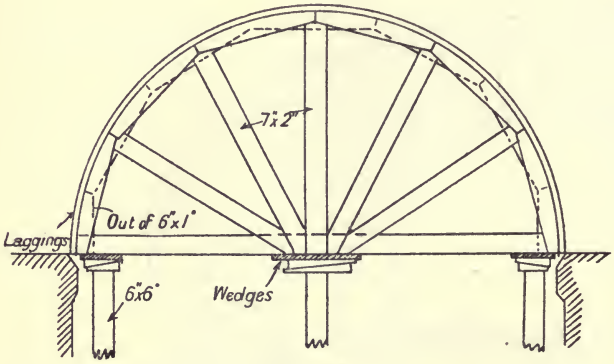


Fig. 152.

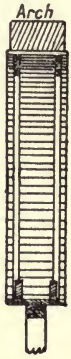


Fig. 153

in the clear of the jambs, the soffit of the arch following this width with a radius of 7 ft.

The centre rests on the projecting architraves of the pilasters, and consists of four frames spaced 9 ins. apart, the whole width of the soffit or concave ring being 3 ft. On this the stone voussoirs of a cut width of 9 ins. were set.

The framing and building up of the centre was an interesting piece of work, the lines of which were laid down on a temporary platform of tongued and grooved flooring boards levelled and trued. On this the joints, radiating lines, and braces were all determined. The battens were not nailed on until the frames, being heavy, were completed, raised to their place with a derrick, levelled and plumbed. It is hardly necessary to dwell on the subject of centres, an

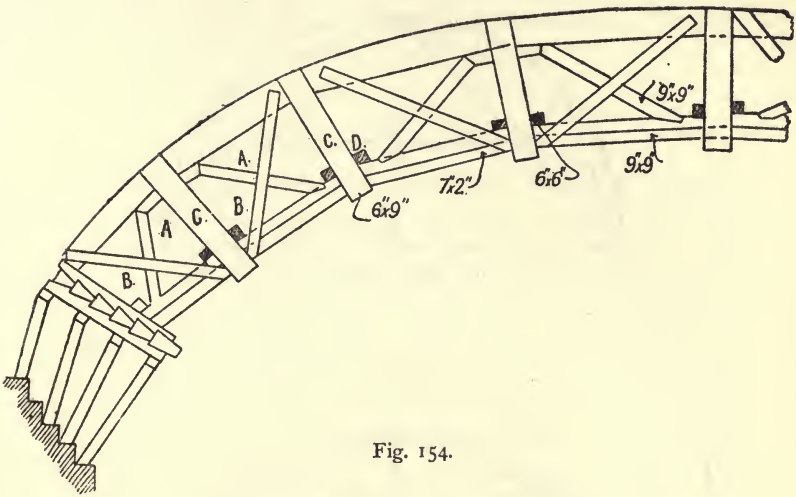


Fig. 154.

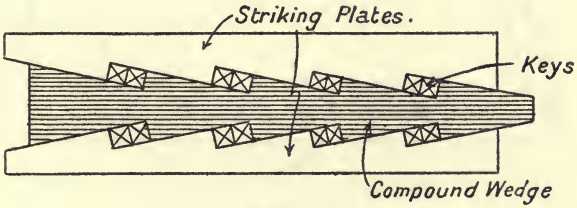


Fig. 155.

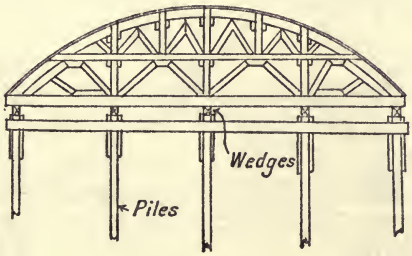


Fig. 156.

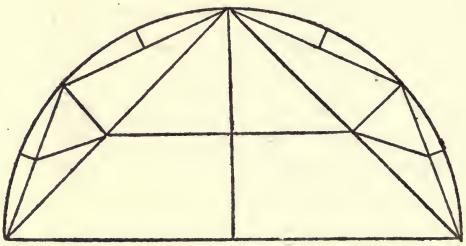


Fig. 157.

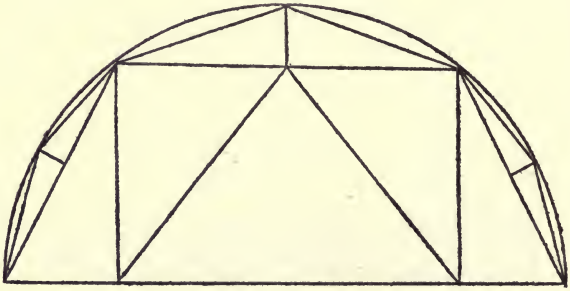


Fig. 158.

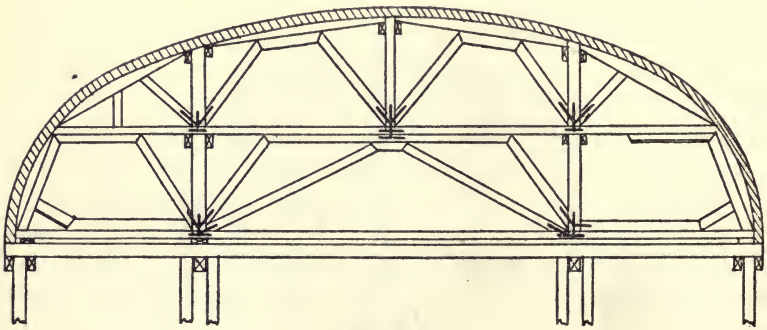
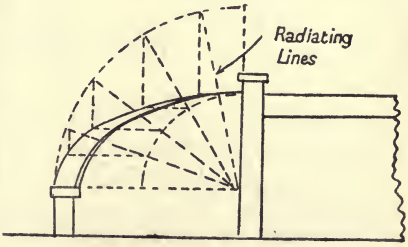


Fig. 159.

infinite variety may be and have been designed, and the engineer will be able to modify any of the above examples to suit any particular case.

Wing Walls of Arches.

It will be seen from the drawings which were previously given, that wing walls play an important part in an arch. They are not merely for appearance, but serve to retain the banks upon which the roadway or railway is carried, and are in consequence retaining-walls, and must be treated as such. They may be straight or curved on plan; the former are the easiest to construct, and the latter give the best appearance, but to execute such a wall, having a straight batter and curved plan, requires care and also the use of a considerable number of templets, because not only have the courses conical beds, but the coping is also twisted. If the plan of the coping is less than a quarter ellipse the side of the pier must be made square with a tangent to the ellipse at the point of intersection with the wing wall. The front elevation will be required on the drawing. The cope of a wing wall curved or plain *might* be a helical or screw surface, but in practice such a wall will not be high enough at the centre of slope to fully protect the bank, and in consequence a conical section must be used, which is a quarter of a circle on plan and shows a quarter ellipse in elevation. Referring to Fig. 160, the plan and elevation of the piers may be drawn. The dotted line construction is then laid down and consists of two quadrants divided into any number of equal parts; on the radiating lines the little triangles (right-angled) are set out, and the apices of these triangles are points on the curve in elevation. In the case of a battering curved wing wall the beds of the



Elevation.
Fig. 160.

courses will dip at right angles to the face, while their front edges lie in horizontal planes. A surface of operation (*vide* Chap. II) must be found on which a curved templet cut to the radius of the front arris is applied. The bevel is cut as shown in Fig. 161, the angle θ being the dip of the bed, while bc=width of stone to be worked and ab=length of stone, de may be made say 3 ins. This templet is applied to all the courses. For each course a curved templet is made to the radius of the first arris and of a length a little more than the longest stone in the course. With these two rules* the top bed of the stone is brought to its proper wind and sunk till the upper edges of both rules are out of winding. Next, each course will require a flexible templet cut to angle *abc* in Fig. 162, and when laid flat will be a portion of the development of the conical surface of the bed. It may be bent round the stone to give the direction of the joint. Two templets are used, as before, to each course. They give the top bed of each course to which they apply and the bottom bed of the course above it. The face is worked with a common square applied at the ends of the top bed, and by means of which drafts are sunk at each extremity of the face, and upon these drafts the thickness of the

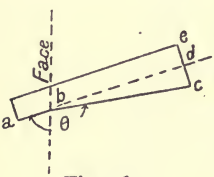


Fig. 161.

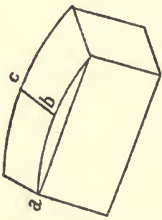


Fig. 162.

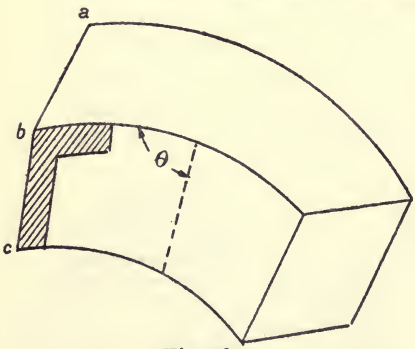


Fig. 163.

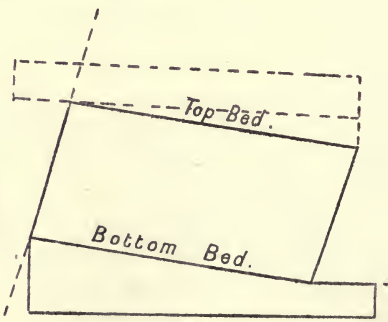


Fig. 164.

* That is two rules like Fig. 161.

course is shown. The face may then be worked between the top and bottom drafts by means of a straight-edge. Referring to Fig. 163, a fourth templet is cut to the angle θ which may be used for one course. The ends of the stones are vertical planes and worked with a straight-edge applied to the arris lines *ab*, *bc*. The bottom bed is worked by the rules shown on Fig. 164 reversed. For laying the stones a radius rod is used, set out as shown in Fig. 165, where *ab* is the vertical height of the wall and *ac* the battering face. At right angles to this each course is marked, as shown, and by the perpendiculars they are transferred to the radius rod,

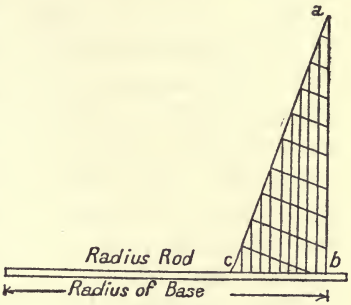


Fig. 165.

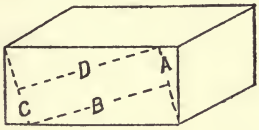


Fig. 166.

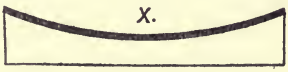


Fig. 167.

which has now the radius of each bed joint marked on it. An iron rod is then set up steady at the centre of the cone. Two batter rules are made with the bed joints marked on

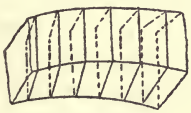


Fig. 168.

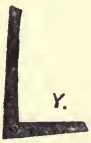


Fig. 169.

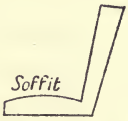


Fig. 170.

same, and they are fixed at the ends of the wall. Then, as each course is laid, its correctness may be tested from the iron rod by the radius rod. It must be most carefully done, as the slightest inaccuracy gives the cope stone a very shaky appearance. To set out the cope stones refer to Figs. 166–170.

At A a draft is worked, then with the templet X the draft B is chiselled, having the same sweep as the bed mould. The draft C is then twisted with A at right angles to B, and finally draft D worked to templet X. Having done this the face is completed by working through B D. The inside face is worked in the same manner to the thickness of the bed mould, and to a templet having the sweep of the bed mould; the result is a stone similar to Fig. 168. The outside face mould is now applied and a draft cut to the joint of the bed mould by bevel Y giving the radiating joint of the stone. Several drafts are now squared through.

Skew Arches in General.—If an arch could be constructed in such a way that the joints would be equal in strength to the actual voussoirs the direction of the courses would have little significance. Although Portland cement has vastly improved the quality of mortar for use with masonry, yet the adhesion cannot be depended on, and in consequence it is usual to place the courses at right angles to the faces of the bridge so that the thrust will act in the right direction, or else failure may occur by the obtuse quoins sliding outwards. In the case of culverts and very small bridges when the load is comparatively light and the span small, skew arches are built in the ordinary manner and cut to a rake on the face, while in the case of culverts are often left square on the face. The generally accepted method for the skew arch is to make it a portion of a hollow cylinder in which the arch stones are laid in parallel spiral courses and with their beds worked in such a way that in any section of the cylinder perpendicular to its axis the lines formed by their intersection with the plane of section shall radiate from the axis of the cylinder. By this method the soffit of each stone will be a portion of a cylindrical surface and the twist of the beds will be uniform all over the arch. Therefore, knowing once for all the angle of twist, the working of the stones is not very much more difficult than with the ordinary square arch. The heading joints will be portions of spirals which intersect the coursing joints at right angles, with the result that the voussoirs are always

square on the soffit, except those on the face of the arch and the quoin stones, the latter differing more or less from the rectangular form according to the angle of skew. It will also be observed from the following pages that the joints of the arch are curved lines, the chords of which all radiate from a point below the axis of the cylinder from which the arch is developed. This distance will depend upon and increase with the angle of skew. A skew arch may be built in many ways, but for the sake of simplicity engineers have usually held to cylindrical arches with spiral courses having beds with a uniform twist radiating from the axis of the cylinder, so that the arch is a segment of or a semicircle on the square and elliptical on the face (or skew). Arches, elliptical on the square, have been built in which the face is simply a flatter ellipse, but the resulting arch is not strong and is not easy to construct.

Arch Stones.—In the case of circular and elliptical arches of which the abutments are square with the face the setting out of the arch stones is a comparatively easy matter. The depth of an arch stone is usually greater than its thickness, and the working of the stone is started on one of the beds; when this is done the ends are set square and the exact shape marked on the stone from a templet and the opposite bed worked to the lines. Finally drafts are sunk at each end of the soffit to the curves previously marked and the soffit dressed off so that it coincides with a straight-edge applied between the drafts in the length of the stone; or the soffit may be formed immediately after the first bed by means of an arch square or curved bevel, and the other bed as worked from the soffit. There is an advantage in the second method that the necessity of squaring the ends before working the soffit is dispensed with. In cutting the stone a straight-edge is used to try the surface between the chisel drafts at the end of the curved soffits. The arch square in the second method takes the place of the templet for getting the chisel drafts.

Setting out Stones for Skew Arches

This is one of the most important works there is in connection with the dressing of stones, and may be done graphically or by the aid of formulæ. Skew arches usually take the form of a cylindrical arch, built in spiral courses, the beds of which will radiate from the axis of the cylinder. The span, rise, width of soffit, and angle of skew must be given and the drawings and diagrams for this work must be drawn to a large scale ; at least 1 in. to 1 ft., because the sizes of the stones will have to be taken from this drawing, which will, if properly done, show the most suitable manner

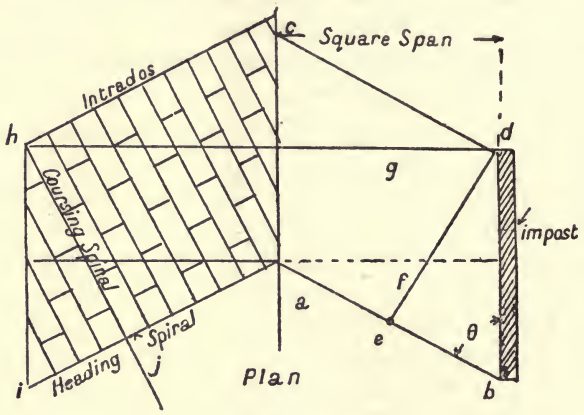


Fig. 171.

of arranging the heading joints in the soffit. A plan of the arch as shown in Fig. 171 must be drawn. *ab* and *cd* are two parallel lines whose distance apart is equal to *de*, which corresponds to the square width of the soffit of the arch. The angle of the arch is set off *abd*, and one impost line *bd*. The second impost line is parallel to the first, and at a distance from it equal to the span of the arch on the square. If the rise=*fg* the square section of the arch can be drawn on the square span line. The development *achi* may then be set out, and upon this should be drawn the development of the heading spiral passing through the extremities of the impost lines in one of the points. Having settled on the

number of stones in each face of the arch this line is divided into that number of equal parts, which must be an *un*even number when there is a keystone. From the point *h* draw the line *hj* perpendicular to the impost line. This will be the development of a coursing joint. This line may pass direct through a division on the line *ai*, but in most cases will not, and the dimensions must be adjusted so that the coursing spiral will pass through one of the divisions. This will either mean altering the width, span, or angle of skew, or all. The angle of skew must usually remain constant, according to the survey plans of the work. If also for some reason the dimensions of the arch cannot be altered then the only thing is to draw this first coursing joint through the nearest division and have the coursing and heading spirals not perpendicular to each other, as they should be, which results in having the soffits of the stones out of square, a decided disadvantage, but a small deviation in the arch dimension such as this would entail is usually permissible. It will only be a few inches in any case. It is for this reason and others that bricks are often preferred for skew arches.

In building small skew bridges of brickwork it is generally considered out of the question to bed each course in the arching to the true curve, since this would necessitate cutting all the bricks, as the traces of successive courses are not parallel to each other. The common alternative of building the courses in parallel equiangular spirals is quite unobjectionable when the breadth of the arch is small compared with its radius of curvature. If, however, the breadth is great, and the angle of the spiral is taken so as to give perpendicularity at the centre of the crown-course trace, then the ends of this course will not be perpendicular to the faces of the arch, and will spoil the appearance of the work, even if their position does not give rise to an objectionable obliquity of pressure.

While maintaining the parallelism of the various courses, and thereby avoiding all bed-cutting of bricks, there is no reason whatever why these should not be approximations —as near as possible—to the true skew-arch course-trace in

other respects. This is effected by drawing one, say the keying-course trace, to the true curve indicated by theory and making all others parallel to it.

It is generally supposed that the calculations necessary for effecting this are elaborate. This is by no means the case. The equation, although it does not admit of a general algebraic solution, can easily be solved with all necessary approximation by one or two trials in a few minutes.

The derivation of the equation may possibly be considered of some interest. It is easily obtained as follows: The crest line of the cylindrical centering is taken as the axis of x, and the curved surface is supposed to be developed in a horizontal plane. The axis of y is taken at right angles to that of x and passing through one extremity of the crest line.

The trace of any vertical plane parallel to the axis of roadway over the arch, on the centering, is an S-shaped curve cutting the axis of x at an angle which is the complement of the angle of skew. Let R=radius of curvature of the centering, θ=angular distance from the crest line, and ϕ=angle of skew. The equation to these lines is

$$x=a+\mathrm{R}\sin\theta\tan\phi \quad . \quad . \quad . \quad (95)$$

$$y=\mathrm{R}\theta \quad . \quad . \quad . \quad . \quad . \quad (96)$$

or if $\mathrm{K}=\tan\phi$

$$x=a+\mathrm{KR}\sin\frac{y}{\mathrm{R}} \quad . \quad . \quad . \quad (97)$$

$$\therefore \frac{dx}{dy}=\mathrm{K}\cos\frac{y}{\mathrm{R}} \quad . \quad . \quad . \quad (98)$$

The perpendicular curve which is the course trace sought is

$$\frac{dy_1}{dx_1}=-\mathrm{K}\cos\frac{y_1}{\mathrm{R}} \quad \text{or} \quad \int\frac{1}{\mathrm{K}\cos\frac{y_1}{\mathrm{R}}}\,dy_1=b-x^1 \quad . \quad . \quad (99)$$

$$\text{or} \qquad \frac{r}{k}\log e\tan\left(\frac{\pi}{4}+\frac{y_1}{2r}\right)=b-x^1 \quad . \quad . \quad . \quad (100)$$

and this is the equation to any skew-trace course. In the particular case of the central or keying course trace, if A=half the breadth of the arch on the skew, x=A when $y=o$ $\therefore b$=A. Therefore to lay out the crown-trace course

in all ordinary cases, it is quite enough to calculate the ordinate at the end of the arch face. This is, of course, the intersection of the above curve with

$$x = \mathrm{KR} \sin \frac{y}{\mathrm{R}} \quad . \quad . \quad . \quad . \quad . \quad (101)$$

or $$\log e \tan\left\{\frac{\pi}{4} + \frac{y}{2\mathrm{R}}\right\} = \frac{\mathrm{KA}}{r} - \mathrm{K}^2 \sin \frac{y}{\mathrm{R}} \quad . \quad . \quad (102)$$

This equation may be easily solved by trial, taking as a first approximation a value for y less than if the curve was drawn in the ordinary way by prolonging a perpendicular to the centre line of the roadway at the crown. Having found y it only remains to measure it off round the convex surface of the centering to obtain the point at which the end of this crown-course trace meets the face of the arch.

Perpendiculars to the vertical-plane traces at these three known points, being tangents to the curve, and but slightly inclined to each other, enable the trace to be drawn without difficulty.

In this way any of the true course-traces may be obtained. If more than one are used it is best to calculate the two which pass through the obtuse angles at springing. In this case, however, cut bricks or cut keystones would be required for closing in at the crown.

Alternative Method.—Given the obliquity of the arch and the number of voussoirs required for its construction, the figure of the stones may be ascertained by the following graphical construction. *Vide* Fig. 172.

Ascertain the centre C of the span A B which connect with the points of division or joint lines of the arch by straight lines as shown. Again, at the point C make the angle A C G equal to the given obliquity. Take any point H, and from H draw H J perpendicular to the span line A B, and meeting it in J; on J C describe the semicircle J L N C, cutting the joint lines produced in K L M and N. Now with the distances J K, J L, etc., describe arcs intersecting J C in the points K′ L′ M′ and N′, join K H′, H L′, H M′, and H N′, then the resulting angles H K′ A, H L′ A, etc., will be the angles of the faces of the stones to which they refer and shown by thick lines.

In order to find the angle of the bed on H C describe the semicircle H M^2 C, then from C, with the distances C K, C L, etc., as radii, cut the semicircle H M^2 C in the points K^2, L^2, M^2, and N^2, join the points H K^2, H L^2, etc., then H C K^2, H C L^2, H C M^2, and H C N^2 will be the angles of the bed.

Referring to Fig. 172, each impost must be divided into

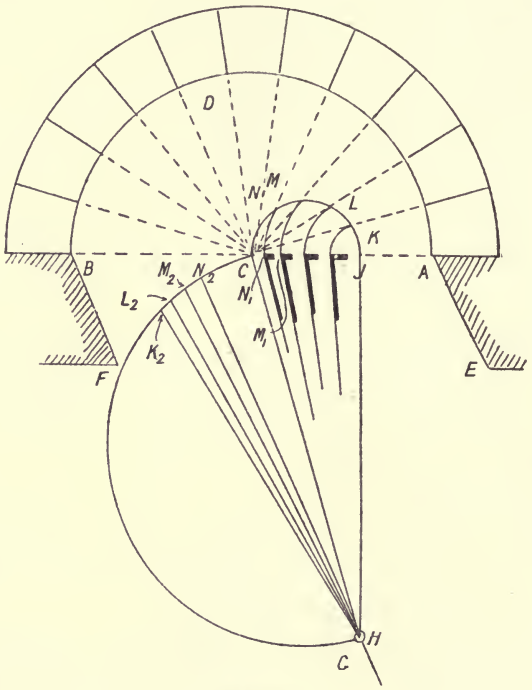


Fig. 172.

as many parts as there are divisions cut off on the heading spiral by the coursing joint first drawn, and through these divisions and those on the heading spiral the developments of the coursing spirals must be drawn, which will be equidistant and parallel. Through the impost divisions also heading spirals parallel to the first must be drawn, and the heading joints on these lines so arranged as to form a regular bond all over the whole soffit. The heading spirals

will not be parallel to the face line, and the quoin stones will in consequence be of irregular length. This may be avoided by drawing the development lines parallel to the face lines, at distances equal to the proposed lengths of the long and short quoins, and to make the end of each quoin a portion of a separate heading spiral passing through the intersections of these lines with the coursing joints. This construction is necessary especially in the case of brick arches with stone quoins, otherwise the ends of the quoins will be out of square with the brickwork. In order to receive the ends of the courses the imposts will have to be cut into skewbacks. They will be square to the axis of the cylinder from which the courses radiate and also to the faces of the abutments.

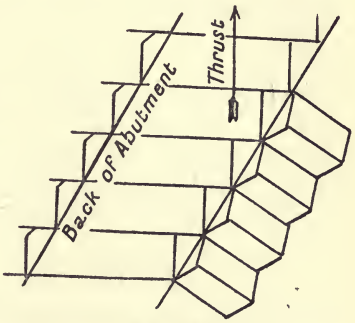


Fig. 173.

It must be remembered that the stone at the quoin, making an obtuse angle, is wider at the back than at the front, while with the other quoin obviously the reverse is the case, and care should be taken that these stones are not too small for proper bond. The backs of the imposts should be cut to bond with the rest of the masonry, as shown in Fig. 173, because the thrust of a skew arch, when properly built, is not at right angles to the abutments, but parallel to the face, and in consequence the joints should be square to the face. If it is required to draw an elevations of a skew arch, it must be borne in mind that the curves are segments of ellipses, the span of which is taken from the plan and the height, from the section, while the positions of the intrados joints are taken from the divisions of the face line of the development of that intrados. To plot out the joints on the extrados it must be taken into consideration that as they radiate at right angles to the axis of the cylinder, and as the length of the extrados is greater than the intrados, therefore the angles made by the abutments with the spirals of the former are greater than those of the

latter. As shown in Fig. 171, the plan of the extrados is out of square; when the heading spirals in the development of the extrados are drawn they will not be at right angles to the coursing spirals and will fall within the face at the obtuse and beyond it at the acute quoins in place of through the intersections of the face and impost lines. In the drawing the extreme heading spirals are divided into as many equal parts as the heading spirals of the intrados, and through these divisions the developments of the extradosal coursing joints are drawn. The divisions on the face line of the extrados are transferred to the curve of the extrados in elevation, and the face joints drawn in between the points on the extrados and intrados so found. The face joints will not be *true* straight lines, but this is usually disregarded on the drawing where the angle of skew is only moderate.

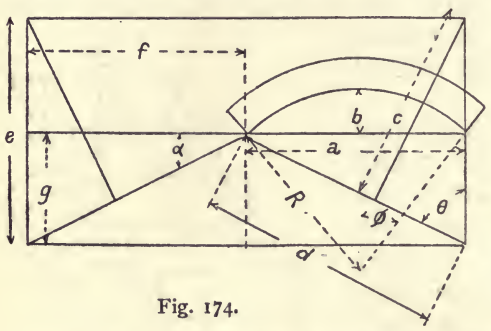


Fig. 174.

Finally the length of the quoin stones and depth of the spandrel course have to be ascertained. The latter will diminish from the springing upwards to the crown, not necessarily for constructional but rather for appearance sake, and in consequence the lengths of the quoin stones must be adjusted. This can be obtained from the drawings showing the elevation when very carefully done. The bevels and templets may be found by simple formulæ.

Let θ=angle of skew,

,, a=square span,

,, b=versed sine of arch,

,, c=width of soffit on the *square*,

referring to Fig. 174.

$$\text{Radius } R = \frac{\left(\frac{a}{2}\right)^2 + b^2}{2b} \quad . \quad . \quad . \quad . \quad (103)$$

Oblique span $d=$ $a \operatorname{cosec} \theta$ . . . . (104)

Length of impost $e=$ $c \operatorname{cosec} \theta$ . . . . (105)

Natural sine of $\angle \phi=$ $\frac{\frac{a}{2}}{R}$ . . . . (106)

Then $f=$ $(2 \phi R) \cdot 017452$. . . (107)

When ϕ is given in the number of degrees

$$g=a \cot \theta \quad . \quad . \quad . \quad . \quad (108)$$

$$d=\sqrt{g^2+f^2}=a \operatorname{cosec} \theta \quad . \quad . \quad (109)$$

angle of intrados $=$ $\frac{g}{f}=\sin \alpha$ . . . . (110)

d also may be found equal to

$$\frac{a}{\sin \theta} \quad . \quad . \quad . \quad . \quad (111)$$

and $e=$ $\frac{c}{\sin \theta}$ . . . . (112)

When the dimensions are so found the lengths of the impost and developments of the heading spiral are set out very exactly on long rods and divided into the number of equal divisions previously determined. The divisions of the impost rod will give the exact length of the skewbacks to be cut on the springers and the divisions on the other rod will show the exact width of the courses.

Centering for Skew Arches

The centres upon which skew arches are constructed will be different from those for square ones, and are a little more difficult to make because the ribs should be placed parallel to the face and not to the centre line of roadway. The laggins have to be very carefully fixed and planed up true to templet cut to the curve of the soffit, and upon this depends the accuracy of the work. The laggins are made close, and upon them the coursing and heading joints are marked. To do so the face lines are drawn and bisected, and a bevel line drawn along the crown of the centering from centre to centre of each face. Upon a long, thin straight-edge the divisions of the heading spiral are transferred from the rod

previously prepared as stated. The length of the straight-edge is equal to the distance from impost to crown of arch. Starting at the end of one of the imposts the flexible straight-edge is bent round the centering and a series of heading spirals drawn from impost to impost through the divisions on the centre line and corresponding lines on the checks on the springing stones. The divisions on the straight-edge are transferred to the heading spirals, making sure that the centre line at the crown passes through the centre of a division in every case. Having thus found these points, the coursing spirals are drawn, which will coincide with the coursing joints in the soffit of the arch. Finally the heading joints are marked and the number of each arch stone chalked on to facilitate laying. The face joints, unlike the heading joints, are not perpendicular to the curve of the soffit, but will make various angles with the coursing joints.

Arch Stress.—For the calculation of flat arches, that is, with a rise of less than one-quarter the span, the following method of determining the stresses with a very near approximation to the truth is useful when higher mathematics are understood by the reader. The thrust is determined from the equation

$$\int Myds = 0 \quad . \quad . \quad . \quad . \quad (113)$$

an equation expressing the invariability of the span, the integration extending from one abutment to the other ds being substituted for dx as an approximation, and y being known as a simple function of x by the equation to the parabola the equation is immediately integrable.

Let l=span and v=rise. The equation referred to one abutment as origin is

$$y = \frac{4v}{l^2} x(l-x) \quad . \quad . \quad . \quad (114)$$

and for a number of loads P at points x y, the horizontal thrust T

$$= T = \frac{5l}{32v^2} \Sigma Py + \frac{5l}{128v^3} \Sigma Py^2 \quad . \quad (115)$$

If a curve whose ordinate is y_1 is described, and

$$y_1 = \frac{y^2}{4v} = \frac{4v}{l^4} x^2(l-x)^2 \quad . \quad . \quad (116$$

in the lower side of the axis x, this curve will be the auxiliary curve. Let g=centre of gravity of P applied at points on the parabola above x and g, the same for values of P applied at points on this auxiliary curve with the same abscissæ, then gg_1 is a vertical line and

$$T=\frac{5P}{32}\times\frac{l\times gg_1}{v^2} \quad . \quad . \quad . \quad (117)$$

By working out a series of values for $\frac{y}{v}$ and $\frac{y_1}{v}$ for any values of $\frac{x}{l}$ the curves may be easily traced. The maximum value of T from a series of loads whose mutual horizontal distances are constant is produced in such a position of the loads as brings their centre of gravity in the middle of the span. For any uniformly distributed load pa covering a horizontal length d of the span from one abutment

$$T=tn^2\left[\frac{5}{2}(1-n^2)+n^3\right] \text{ or } t\phi(n) \; . \quad (118)$$

where t=horizontal reaction for a load pl covering the span and $n=\frac{a}{l}\phi(n)$ may be worked out on a series of values for reference. For the same loading the BM throughout the span are represented by two parabolas having a common point and tangent at $x=a$ (where the load ends). These parabolas can be drawn to any value of a. The envelope of these parabolas for various values of a is a curve whose ordinates represent the max. BM at various points of the span as the load progresses. The maximum values of BM occur at one-quarter and three-quarter span, and

$$\text{BM max.} =\frac{1}{60}pl^2 \quad . \quad . \quad . \quad (119)$$

The shearing face$=\frac{dM}{ds}$ when $\frac{dM}{dx}$ is used as an approximation, the maximum shear F occurs at the abutments and springing, at the former when the load is on three-tenths of the span and at the latter when on one-half span. For any single load P, the uniformly distributed load which would

give the same maximum compression as P the distributed load pl can be approximately expressed by

$$pl(1{\cdot}1+0{\cdot}007\frac{l}{v}) \quad . \quad . \quad . \quad (120)$$

The foregoing paragraph is deduced from a paper by Mons. E. Collignon in *Annales des Ponts et Chaussées*, April, 1890, to which the reader is referred for further information.

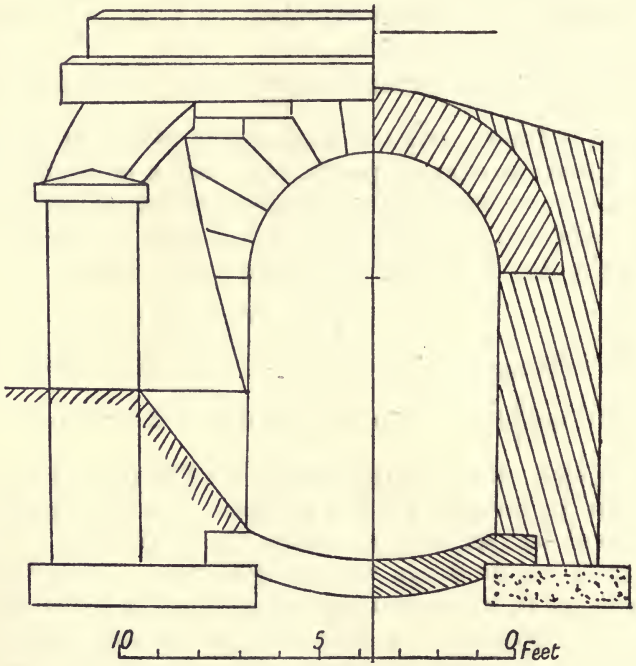


Fig. 175.

It is assumed by the author of the paper that as the circle differs so little from the parabola in the case of a flat arch, that the above reasoning which refers to parabolic elastic arches may be safely used for circular ones.

Culverts of large size are really small masonry bridges. In the case of sewers and long culverts of comparatively small size and great length, bricks and concrete are principally used, and their construction is a subject which belongs

to books on sewerage. It is discussed fully in the author's *Main Drainage of Towns*. Culverts of short length are those which would serve to carry water under a road or railway or reservoir bank. In the latter case the load they have to carry is that of the bank immediately above them, and in the former case is one which is subject to such a variety of contingencies that no definite calculation of the strength can be made. Ordinary types of masonry culverts are shown in Figs. 175 and 176, about which there is no special construction; such culverts should be solidly

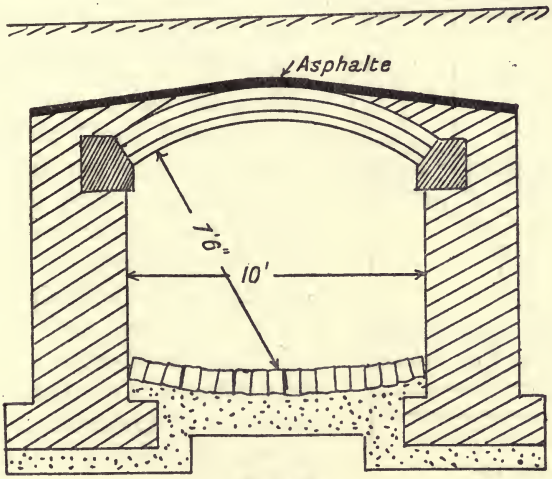


Fig. 176.

built and of such a length that the bank above them will rest at its ordinary angle of repose and not have any thrust on the parapet wall. They should provide an ample waterway for the maximum floods which in the case of small freshets are often vastly in excess of the ordinary flow, and the bottom should consist of a curved invert of Stafford blue bricks laid in cement or roadway setts, both laid as stretchers parallel to the centre line in plan. A form of culvert designed to give a large waterway for a small quantity of material is that shown in Fig. 177. A diagram for the strength of the same is shown.

The load is assumed as earth weighing 100 lbs. per foot

cube and a live load of $3\frac{1}{2}$ cwts. per foot super. The maximum stress occurs across space 3, where

$$\frac{W}{A}\pm\frac{M}{Z}=\frac{150}{1\cdot9}\pm\frac{150\times\cdot8}{\frac{1}{6}(1\times1\cdot9^2)}=79\pm200$$

$=279$ cwts. compression and 121 cwts. tension—respectively to 13·95 tons and 6·05 tons per square foot. These stresses are somewhat high, and the arch should be thickened up as shown, and if the values of W+M are again worked out the values will be

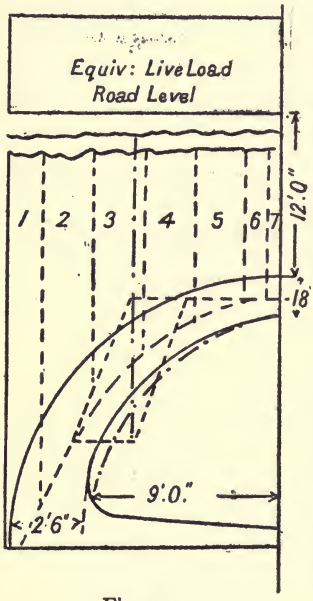


Fig. 177.

$$\frac{150}{2\cdot2}\pm\frac{150\times\cdot6}{\frac{1}{6}(1\times2\cdot2^2)}=68\pm112$$

$=180$ cwts. compression and 44 cwts. tension equal respectively to 9 and 2·2 tons per square foot.

An ordinary form of culvert suitable for a clear waterway of 10 ft. and having a flat arch necessary in a low bank is shown in Fig. 178, the arch being of three rings of $4\frac{1}{2}$-in. bricks. The outlet culverts of earthen reservoir banks are of importance. They may be constructed in the actual puddle bank, or may tail into the bank round the ends of the puddle wall. The reason for this obviously expensive construction is that less disturbance is likely to occur to the puddle wall, and hence less likelihood of failure and leakage, and also the continual settlement of the puddle is at all times likely to cause fracture to the masonry or brickwork culvert, and it is the opinion of many engineers that it should be carried round the ends of the wall. It is in this case that the by-wash is required, but when the outlet culvert is in the middle of the bank it is more usual to

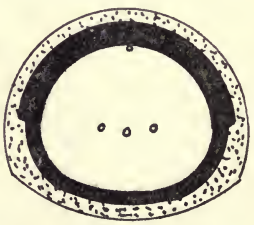


Fig. 178.

construct it once for all and allow all water to flow through it. At the same time the centre of such a culvert should be supported not on the mere puddle, but a solid wall of masonry or concrete. The use of slip joints on the culvert are not to be advised.

The shape of such culverts is sometimes circular; theoretically, an elliptic section of ratio of horizontal to vertical axis of 2 to 3 is best, the bottom being usually a short inverted arch. They have been, however, constructed of all sizes and shapes. They must in all cases be very carefully backed with puddle, or preferably Portland cement concrete. They are widened out at each end into two bays with wing walls, called the "fore bay" and "tail bay" respectively, according as the inner or outer end is referred to. And if we have d=diameter of culvert when circular, and the smaller diameter when elliptical, and H=height of earth above it in feet, we can find the correct thickness of the masonry by the following equation, the result being given in inches, d being in feet,

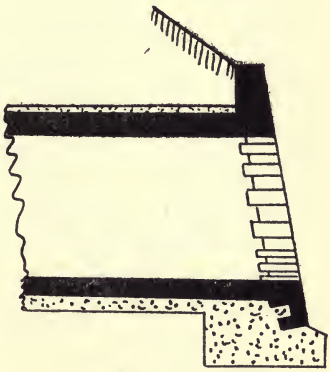


Fig. 179.

$$t=d\sqrt{\frac{H}{2}} \quad . \quad . \quad . \quad (121)$$

A typical culvert section is shown in Fig. 179. The pipes are then all laid in this culvert, and it should be of ample size for repairing them too, and either a valve tower is provided or a masonry diaphragm (or "stopping") built into the culvert when it is desired to fill the reservoir, the former being the usual modern practice. It should be borne in mind that the fore bay and tail bay are not merely structures, as they may at first seem, for ornamental purposes only, but they have to withstand a large thrust of the embankment and must be treated as surcharged retaining-walls.

Tunnels.—The subject of making large tunnels is a

study in itself, but the lining of the tunnel and the entrance structure belong to the art of the mason. In rock it is not usual to line the tunnel except in places where a slip is likely to take place. The tunnel entrance is a piece of work into which artistic effect is sometimes put, but modern practice tends to simplicity in this direction. As the tunnel will usually be approached by a cutting, the face of the tunnel, like a reservoir culvert, must be treated as a surcharged retaining-wall, while wing walls will be required to keep the side banks in place. They may be curved or straight, and although the design of a tunnel entrance will be according to the individual ideas of the designer, yet

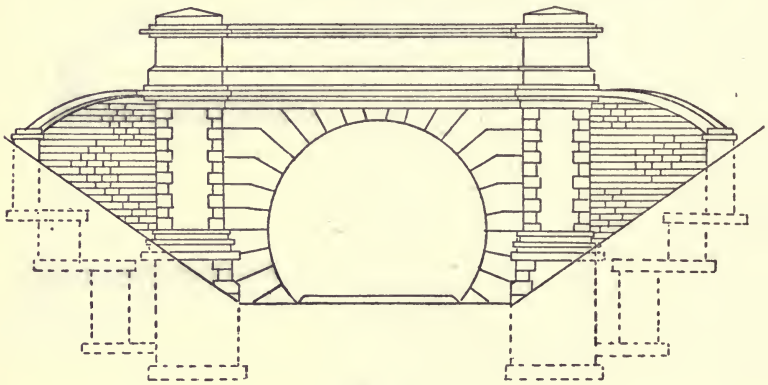


Fig. 180.

Fig. 180 is given as a typical example of a simple, inexpensive, and yet effective piece of work. The arched lining of a tunnel is usually at a depth below the surface many times the radius of the arch, and in consequence the elliptical arch is the one which gives the proper form for the line of pressures, and in which the ratio of horizontal to the vertical semi-axis is the square root of the ratio of the horizontal to the vertical pressure of the earth, or if $\phi = \angle$ of repose and H=horizontal semi-axis and v vertical semi-axis, then *

$$\frac{H}{v} = C = \sqrt{\frac{P_y}{P_x}} = \sqrt{\frac{1 - \sin \phi}{1 + \sin \phi}} \quad . \quad . \quad (122)$$

* See paragraph on arches.

The value of C must then be less than that given in the foregoing equation, or else the sides will be forced inwards. In railway tunnels it is usual to have the sides and roof equal to two-thirds or three-quarters of the ellipse and an invert in the form of a segmental arch, as shown in Fig. 181, which is a section of the Bath Tunnel, G.W.R., and is a typical one. For tunnels conveying water the whole ellipse may be used. The curvature of the invert when segmental should be one-eighth the span. By this method of construction (ellipse and inverted arch invert) the vertical pressure on the sides of the tunnel is concentrated upon the foundation course. The duty of the invert is to prevent the sides being forced in under the earth pressure laterally; in fact, the greatest intensity of pressure in a buried archway such as a tunnel will occur in the sides at the ends of the shorter diameter of the oval intrados.

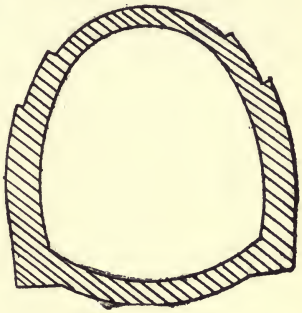


Fig. 181.

If d=depth of this diameter below ground surface, and s=half-span of the archway, v=rise of archway, and t thickness of the side, the greatest pressure in lbs. per square foot=

$$P=W\frac{[d(s+t)-\cdot 8vs]}{t} \quad . \quad . \quad . \quad (123)$$

W being the weight of 1 cube foot of the earth. It must be borne in mind that in tunnels deep under the ground the load is dead and the factor of safety may be low, but in the case of culverts under railways and roads there is always the live load which, so far as its intensity goes, cannot under such circumstances be exactly calculated, but a factor of safety of 10 is provided to guard against failure, that is to say, the thickness of the culvert must be sufficient to give a value of $p=\frac{P}{10}$ found by Equation 123. If s and v

are as before the longest radius of the intrados=

$$R=\frac{v^2}{s} \quad . \quad . \quad . \quad . \quad (124)$$

and the least value of t, which must satisfy $\frac{P}{10}$ in the case of culverts, etc., under low banks and $\frac{P}{5}$ in the case of deep tunnels

$$=t=\sqrt{\cdot 30\,R} \text{ in feet.} \quad . \quad . \quad (125)$$

As shown in Fig. 181 t is variable, and may be made so in the same ratio as s to v.

CHAPTER VII

MASONRY TOWERS AND PILLARS

Masonry Towers.

THE term tower, as considered in this chapter, is held to represent a structure in which the main stress will be that due to the wind, and in the case of lighthouses the waves. A tower may be such a structure as a walled tower, a tower for ornamental purposes, or for the purpose of raising something to a permanent elevation, such as a clock, a lighthouse, a monumental column, a short tower or a chimney. The last-named are rarely built of masonry, but of brickwork or concrete, but the stresses are precisely the same as considered in this chapter. The author only knows of one chimney built of stone, and that is at the Ballycorous works of the Mining Company of Ireland. It is an interesting structure, having a spiral stone stairway winding round the exterior. It is a familiar landmark in Co. Dublin as it stands high up on a hill.

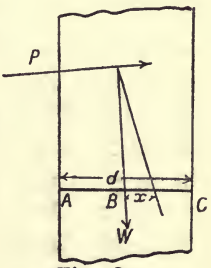


Fig. 182.

There are various maximum values of wind pressure in Britain used by engineers. In such works as towers, in which the wind plays the most important part in the stresses thereon, a maximum pressure of 56 lbs. per square foot is advisable. Special values are taken in foreign countries, according to statistics or experiments.

For the general stability of towers the reader is referred to Fig. 182.

Now in any tower let a=the area of a *vertical* section across any diameter and p the greatest wind pressure.

The total pressure = P.

For square towers $P = pa$ . . . . . (126)

For round ,, $P = \frac{pa}{2}$ . . . . . (127)

—in accordance with usual practice.

Then taking the moment of pressure about any horizontal joint at h feet above that joint, the moment in lb.-ft. or tons = inches according to units taken = hP.

Square towers $hP = hpa$ . . . . . (128)

Round ,, $hP = \frac{hpa}{2}$ . . . . . (129)

which must be checked over by the formula $\frac{W}{A} \pm \frac{M}{Z}$ to find out if the tension and compression on the joints are in safe limits. Obviously the higher the column the greater will be the necessary strength of brickwork, due both to extra load and extra wind pressure. The foundations also will be materially increased. Failure to comply within safe limits with the condition $\frac{W}{A} \pm \frac{M}{Z}$ will cause two things —crushing of joints on the leeward side, or more likely opening of them on the windward. When a crack is so started it will usually extend, but its form is rather peculiar, inasmuch as it extends diagonally down each side of the chimney in zigzag form, following the lines of the joints, of course. But this weakening of the joints on the windward side finally sends so much pressure on those on the leeward side that failure does *finally* take place, due to crushing of the brickwork on that side. Final collapse is, however, usually in the form of a complete breaking up. Let us take a chimney out of the perpendicular, as in Fig. 182. Let d = its diameter as shown, and let x = the deviation of its centre from the original perpendicular centre line. * Let q = ratio of x to d, while q' = ratio of the deviation of the point B from the middle of the diameter A C.

* The value of q for square towers is equal to $\frac{1}{6}\left(\frac{1+d^2}{D^2}\right)$, and if d and D = inside and outside diameters respectively, $\frac{1}{8}\left(1 \pm \frac{d^2}{D^2}\right)$, for round ones.

Then $$Wx=(q-q')Wd \quad . \quad . \quad . \quad (130)$$

For practical purposes, however, q=one-quarter for round towers, and, for sake of comparison rather than design, =one-third for square ones.

Reverting to Equations 128 and 129 we have

$$hpa=\tfrac{1}{3}-q'(Wd) \quad . \quad . \quad . \quad (131)$$

and $$\frac{hpa}{2}=\tfrac{1}{4}-q'(Wd) \quad . \quad . \quad . \quad (132)$$

It still, however, remains to find W.

Taking masonry at 112 lbs. per foot cube=wa as before, B=mean thickness of the masonry just above any joint under consideration.

Then for round chimneys,

$$W=3{\cdot}141Bwd \quad . \quad . \quad . \quad (133)$$

and for square ones, $$W=4Bwd \quad . \quad . \quad . \quad (134)$$

and combining with previous equations for square towers,

$$hp=\left(\frac{4}{3}-4q'\right)Bwd \quad . \quad . \quad (135)$$

and for round towers,

$$hp=(1{\cdot}51-6{\cdot}28q')Bwd \quad . \quad . \quad (136)$$

The thickness of masonry to ensure stability can then be found.

Graphical Methods.—By referring to Fig. 182*a* the matter can be treated graphically. Let A B be the width of the

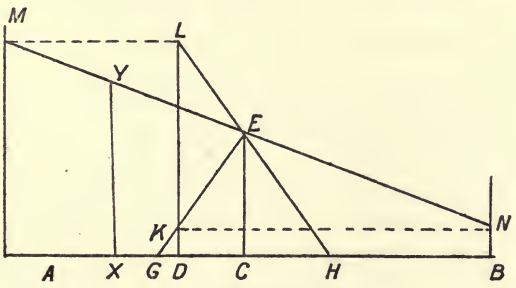


Fig. 182*a*.

foundation, C the geometrical centre, and D the point at which the resultant, due to the wind pressure and the weight, cuts the base. Through C set up a line C E equal, to any convenient scale, to the mean pressure on the foundation—i.e.

the total weight of the chimney and footings divided by the area of the foundation. Now mark off points G and H on either side of C, and make C G=C H=$\frac{1}{6}$ A B. Join E G and E H. Draw a vertical line through D, cutting E G and H E produced in K and L. Draw the horizontal lines K N and L M, cutting the verticals through A and B at M and N. Then the ordinate as X Y between A B and the line M N gives the intensity of pressure at any point to the same scale to which C E gives the mean, or average, pressure. A M therefore gives the maximum pressure.

As an example of the method of applying these rules we will calculate the safe wind pressure and the maximum compressive stress at the base of a tower 160 ft. high, 9 ft. square inside at the top, and 11 ft. square inside at the base. The thickness is 13$\frac{1}{2}$ ins. at the top and 49$\frac{1}{2}$ ins. at the base. The base is consequently 19 ft. 3 ins. square outside, and the top 11 ft. 3 ins. square outside. The mean section is 15 ft. 3$\frac{1}{2}$ ins. square outside and 10 ft. square inside, and the volume of masonry=V=$160(15{\cdot}29^2-10^2)$= 21,300 cub. ft. We may take the weight of masonry as 112 lbs. per cubic foot, and the total weight of masonry is therefore $112\times21{,}300=2{,}380{,}000$ lbs. The area of the base is $19{\cdot}25^2-11^2=249$ sq. ft., and the mean pressure works out at 9600 lbs. per square foot.

When the thickness is small compared with D, the ratio $\frac{d^2}{D^2}$ becomes equal to 1, and the values of q are one-third and one-quarter for square and round sections respectively. Rankine (*Applied Mechanics*, p. 229) states that these values are sufficiently accurate for practical problems relating to chimneys. As the error is on the wrong side, however, and is considerable especially in chimneys of small diameter, it would seem safer to use the formula above. The other extreme case is where d=0—i.e. when the bed-joint is a rectangle or circle and not an annulus; for this case q=one-sixth for square sections and one-eighth for round sections. This particular case occurs in determining the distribution of pressure on the foundations.

Stability.—We will now proceed to find the greatest distance

from the centre at which the resultant may cut the base consistent with there being no tension at the windward edge. This is $19{\cdot}25 \times \frac{1}{6}\left(1+\frac{121}{270}\right)=4{\cdot}25$ ft. By taking moments about this point we can find what the wind pressure will be when the resultant passes through this point. The weight acts vertically through the centre, and the weight moment is therefore equal to $2{,}380{,}000 \times 4{\cdot}25$. The wind moment is equal to the wind pressure multiplied by the area of a side multiplied by the height of the centre of gravity of this area above the base. The height of the centre of gravity in this case is 73 ft., and the wind moment is therefore $P \times 160 \times 15{\cdot}29 \times 73$, where P is the pressure in pounds per square foot. Equating the wind moment to the weight moment we have $P=\frac{2{,}380{,}000 \times 4{\cdot}25}{160 \times 15{\cdot}29 \times 73}=56{\cdot}5$ lbs. per square foot. A wind pressure of 56·5 lbs. per square foot is therefore the maximum which this chimney would bear without producing tension at the base, and the maximum compression with this wind pressure would be $9600 \times 2=19{,}200$ lbs. per square foot.

From an engineer's point of view the principal use for a masonry tower is to support something at an elevation, such as a water-tank, in which case the tower is called a water tower. There are many examples of such towers, large and small, in England. There is a large one at Chester and Norton, in Cheshire, both of which can be seen on the L.N.W.R., Liverpool and Holyhead section. In such cases wind pressure is quite a minor detail, as the masonry necessary to support the load added to that load will provide a value of W in Equations 133 and 134 far in excess of that required to resist the wind alone. These towers are generally designed from an artistic point of view, of which Fig. 183 is given as an example. They are now frequently made of reinforced concrete and steel or cast iron.

Lighthouses are structures which will always remain masonry structures when of any size or importance. They constitute the most solid structures which the engineer is called on to design, and are often executed under the most

trying and hazardous conditions. Not only has the force of the wind to be considered, but also the force of the waves,

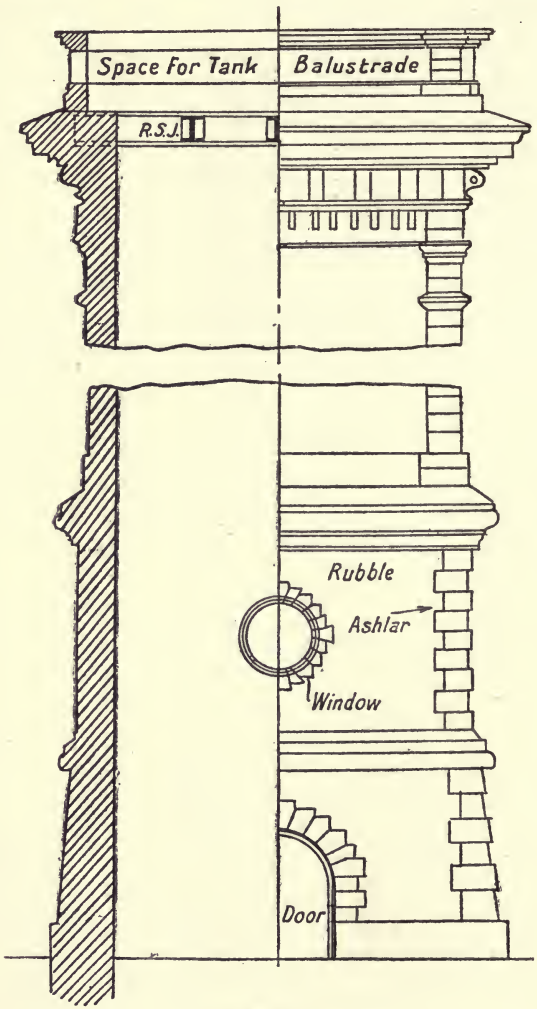


Fig. 183.

which is a force of considerable moment and one which has successfully destroyed man's handiwork more than anything else. The force of the sea can be measured with

more or less accuracy at any particular place by dynamometers. The construction of such apparatus is out of the scope of this work, but is discussed in books dealing with sea defence works and in *Lighthouses*, by Stevenson (Spon). On the force of the sea some experiments have lately been carried out by Mr. Dick of the Engineers' Department, Irish Board of Works, and he writes as follows :—

"I am led to the conclusion that the force diagram should be composed of straight lines rather than of parabolas,* and that the zero pressure below should be taken at a depth of two and a half times the wave height instead of twice.* I think a fair depth of wave velocity would be that at 250 ft. from wall—viz. 16 ft. This with 5-ft. wave gives $13\frac{1}{2}$ ft. as depth below trough of wave, and the calculation of pressure at mean level of wave (H W S T) would stand as follows :

$$V=\sqrt{2g(\tfrac{1}{2}\times 13\tfrac{1}{2})+(\tfrac{3}{4}\times 5)} \quad . \quad . \quad (137)$$
$$=8\cdot02\sqrt{10\cdot5}=26 \text{ ft. per sec.}$$

"Allowing for rapid shoaling and occasional interference, this velocity should in this case be multiplied by 1·80, giving an occasional maximum velocity of attack of 46·8 ft. per sec., for which the corresponding value of it is 34·2 ft.

"Force of mean level of wave is,

$$\therefore \frac{2\times 34\cdot2\times 61}{240}=1\cdot95 \text{ tons per square foot} \quad (138)$$

and taking zero force at $2\frac{1}{2}$ height below H W S T the force at dynamometer 7 ft. below H W S T is

$$(12\tfrac{1}{2} : 5\tfrac{1}{2} : : 1\cdot95 :) \cdot860 \text{ ton}=1930 \text{ lbs.}$$

The force actually recorded being 1980 lbs."

Further information on this important subject may be had in *Report of British Association*, 1844. *Cambridge Transactions*, 1842, 1845, and 1850. *Harbours*, by Stevenson. *Edinburgh Transactions*, Vol. XVI. Rankine, *Philosophical Transactions*, 1863. *Transactions of Naval Architects*, 1862.

* "Notes on the Design of Breakwaters," by F. J. Dick, M.Inst.C.E., read before the Inst. C.E. Ireland, 1902.

The generally accepted rule for the height of any waves (maximum) is

$$h=1{\cdot}5\sqrt{d} \quad . \quad . \quad . \quad . \quad (139)$$

where d=*fetch* in miles and h height in feet, d being over forty miles. If under forty miles

$$h=1{\cdot}5\sqrt{d}+(2{\cdot}5-\sqrt[4]{d}) \quad . \quad . \quad . \quad (140)$$

h having been found by experiment to increase in the same ratio as $\sqrt{\ }$distance from windward shore. The force of waves in an exposed situation is always, however, a more or less unknown quantity. At Skerryvore Lighthouse the average force of the waves in summer was found by Mr. Stevenson to be 611 lbs. per square foot, and in winter, 2086, the greatest being 6083. A lighthouse is a tower which must be designed to stand by weight alone rather than cohesion or inertia. Therefore, with this object in view, the centre of gravity of the structure must be as low as possible. The triangular form, of course, will satisfy this contingency in the greatest possible way, but is, obviously, an impossible form in practice; nearest to it comes the frustum of cone, as a triangle revolving generates a cone. This, again, is not suitable as the large angle (nearly 90°) which the side of the tower makes with the foundation under such circumstances is a source of weakness. On these considerations the bare trunk type of structure has been evolved. Ultimate stability depending on the lowness of the centre of gravity, and a monolithic construction, as stated, the cone is the nearest section that would fulfil this, but, as the forces to which the several horizontal sections are subject diminish towards the top in rapid ratio, the form of structure should be evolved by the revolution of some curved line convex to the axis of the tower and gradually approaching parallelism with it. Lighthouses are, when exposed, universally circular in plan, and continue curved in a vertical plan so as to have no abrupt change of outline. There are, of course, many exceptions to this rule. Holyhead Lighthouse is square on plan, Poolbey Lighthouse in Dublin Bay is a sort of peg-top section and looks very solid, but decidedly ungainly. This is not necessary, as a lighthouse may be a thing of beauty and yet equally as

strong as any other form. The height depends on the distance at which the light must be seen, and is decided by the lantern designer. Lighthouses are usually founded on rock; if soft they should spring from the rock by a bell-shaped base having a very wide curve. This will prevent the destruction of the rock by the reaction of the

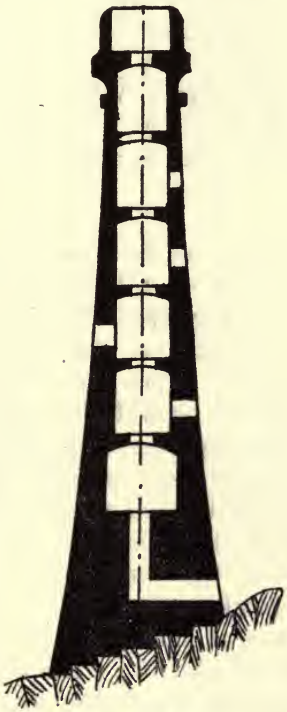


Fig. 184.

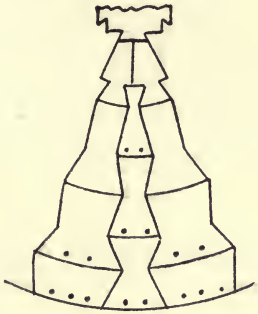


Fig. 185.

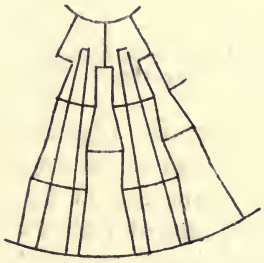


Fig. 186.

waves. All foundations must be carried well into the rock, and in hard rock the connection is left to the engineer, provided that a good design and economical distribution of the materials is obtained. The tower should be placed on that side of the rock which has the most protection from the fetch; depressions, however, are not good places for founding the tower, as they are liable to scour. Level steps must be cut in the rock to receive the stones of the

foundation course, but nothing is gained by tying them down by bolts as may be supposed. Heavy stone should

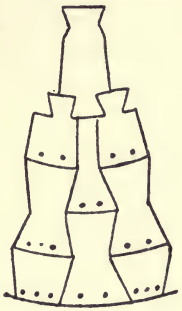


Fig. 187.

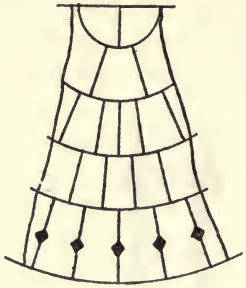


Fig. 188.

be used, as it must be remembered that weight is one of the principal factors.* A general section of a lighthouse is given in Fig. 184. The stones used should be the largest obtainable. They are cut into such forms as will allow them to resist displacement laterally, and they are usually tied vertically by bed plugs. The plan of the foundations of the Eddystone Lighthouse was of the form shown in Fig. 185, that of the original Bell Rock in Fig. 186, and the present structure in Fig. 187, that of the Skerryvore in Fig. 188, and of Duheatach (Mull) in Fig. 189. The floors of a lighthouse are either of vaulting, Figs. 190, 191, or stone slabs, Figs. 192, 193. The door should be on the leeward side and approach the stairway by an archway. The stairway to the first landing is usually winding, the access to the others being by ladders.

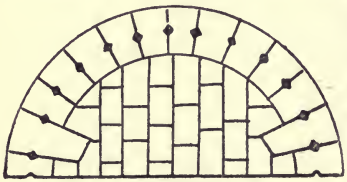


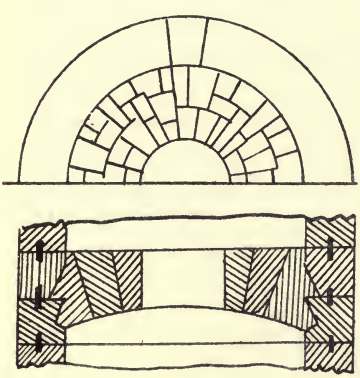
Fig. 189.

Circular stones and vaulting introduce two forms of masonry work not yet discussed.

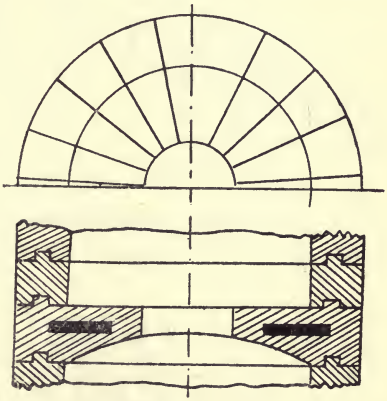
Vaulting is a class of work much used in architecture such as in churches, but, as far as the engineer is concerned,

* *Vide* Report on Eddystone Lighthouse, *Trans. I.C.E.*, Vol. LIII, and Lighthouses in General, Vols. XXX–XLIV and XXXVIII.

is rarely employed. Groined vaulting in brickwork has been employed for the covering of large tanks such as service reservoirs, and the method of setting out and constructing centres for such work is treated in the author's *Civil Engineering Practice*. The vaulting shown in Fig. 190 is plain cylindrical vaulting and does not call for any special construction. The use of flat stones, as shown in Fig. 192, with radiating joggle joints is an equally strong and much simpler construction. The parapet of a lighthouse should be formed of very heavy stones fastened with metal cramps. The overhang should be very small. The whole thing, in fact, in lighthouse work is to so work and



Figs. 190, 191.



Figs. 192, 193.

bind each stone on all its sides with its neighbours in such a way that the whole is one solid mass. If, then, the weight of the whole is sufficient to withstand any possible onslaught of the sea fracture alone will damage it, and with proper bonding in all directions fracture should be impossible. It is the neglect of this that caused the failure of earlier works, and no pains should be spared in making this work as perfect as it is possible to do. The circular stone staircase consists of a series of stones cut in such a way that they form a central pillar. The work is quite simple and will be easily understood from Fig. 194.

Erection.—In building towers and similar structures special

labour-saving appliances are used in place of the ordinary scaffolding. In the case of lighthouses in exposed situations it may be necessary to resort to all sorts of expedients to raise the materials into place, and a description of the various methods employed on former works would be impossible here. Each case has usually to be judged according to its merits by the engineer, who in such cases usually has ample scope for his ingenuity and ability. By reference to the works before given some valuable suggestions may be had by comparing those works with any case likely to be similar.

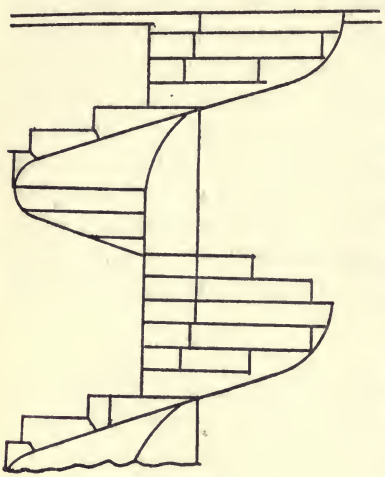
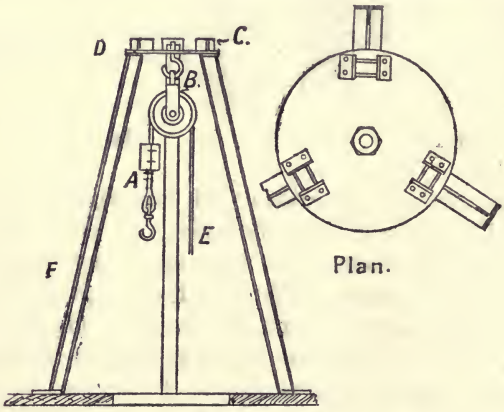


Fig. 194.

Of the simpler appliances for building towers there is the tripod, by means of which the material is rapidly



Figs. 195, 196.

hoisted up the inside of the tower. This is shown in Figs. 195, 196, which is an elevation of the tripod, A being a cast-iron weight, B a cast-iron pulley,

C 3-in. by 2-in. angle-iron brackets, D $\frac{3}{8}$-in. steel head plate, E $\frac{1}{2}$-in. wire rope, F $2\frac{1}{2}$-in. by 2-in. steel T legs, and is so constructed that when the spikes are drawn at the feet the legs can be brought close together. The convenience of this arrangement will be appreciated when the tripod has to be raised from one scaffold to another as the building progresses. The general arrangement is to place the tripod on a series of temporary stages erected on the inside of the tower and fixed to the walls. Outside the tower is an ordinary steam winch, upon which works the rope E by passing through an aperture in the walls of the structure specially left for the purpose. There is a guide pulley attached to the lowermost staging, as shown in Fig. 197.

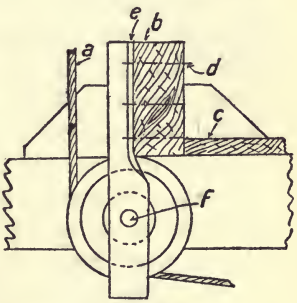


Fig. 197.

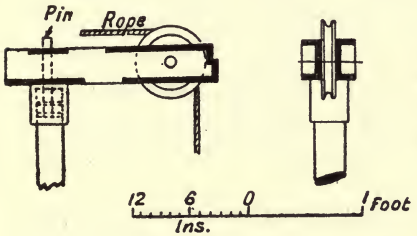
The tripod is constructed of steel T irons and plates, and, in addition to the hinge arrangement already spoken of, provision is made for detaching the legs by the removal of hinge pin at the head. This makes the lowering much easier when the chimney is completed, and is handier for storing when out of use. Instead of fixing the hook to the top plate by riveting the hook bolt, a thread is run for a nut, and above this nut a split pin is inserted into the hook bolt. The split pin prevents the nut being released by the hook bolt turning round with the pulley. The whole tripod arrangement should be designed in such a way that it may be moved from stage to stage with the minimum of trouble. Another system is that of the pole and crosshead. The pole may be of wood and, if so, it must be entirely free from knots or flaws of any kind, the grain running right through the length of the pole; but a galvanized tube 3 ins. in diameter is much better every way than a wooden pole. The construction is the same except that at the bottom the tube is simply flattened out and the groove cut out of it, a sufficiently flat section being left above the groove to form a clutch plate. No protection plate at the centre in this case

is required, but the small pin before mentioned should be retained. At the top the tube is simply plugged up and the pin fixed in the plug. Such a pole has the advantage of not being liable to snap under a sudden strain and lasts longer than a wood one. Details of the pole and crosshead are shown in Fig. 200, and it is made of oak. A bearing plate, which extends 4 ins. on each side of the pin groove, is shown on the bottom of the crosshead, and a protecting plate 5 ins. by 4 ins. is shown on the top. Both these plates, which are fixed with wood screws, have wrought rings which form the top and bottom of the pin groove and prevent the pins damaging the groove. Clearance of $\frac{1}{4}$ in. at least should be allowed, so that should the crosshead be tilted up or down under a load it will not exercise a twisting effect on the pin, and also that the crosshead may swing freely when material is being landed on the scaffold. In some cases the pin is not in the centre of the crosshead, but a little on one side, the object being to counteract the tendency towards the crosshead being tilted up when material is being hoisted, the short arm being on the side of the haulage rope.

The irons are of wrought-iron bars twisted from flat to edge, of $2\frac{3}{4}$ ins. by $\frac{1}{2}$ in. or 3 ins. by $\frac{3}{8}$ in. section, having on their inside end or on the flat end holes drilled at spaces of about 2 ins. or thereabouts for spikes, and on the outside end notches formed so as to suit the clutch and thimble plate of the pole. These notches must also allow clearance for clutch and thimble plates, as any tightness has a loosening effect on the irons, and causes trouble when fitting a new scaffold. The weight is of sandwich pattern, of cast-iron, round in section, with a groove for the rope, and bolted together.

The pulleys (which are shown in Figs. 198–202) are of cast-iron, the pins being cast on and the webs lightened by perforations. In the crosshead they are detachable, being slipped down into position by deep grooves, into which plugs may be fixed in the case of the pulley wheel on the hoisting side of the crosshead. This will prevent the possibility of the pulley wheel being knocked out of the groove should the

weight be run up against it. The guide pulley (above) in which are wrought-iron sheath straps and **V**-tube rollers is fixed to the tower in order to keep the heaving rope in a direct line with the pulley wheel in the crosshead, the pulley being made movable to follow the rope on the winch barrel. There is a small roller hung to this pulley by sheath straps and running close to the pulley wheel. Its duty is to prevent the rope getting fouled when slack. A hand or steam winch is provided, the former being the most usual in this case. A guide rope is usually attached to the



Figs. 198, 199.

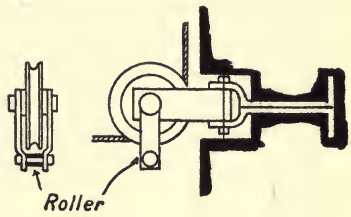
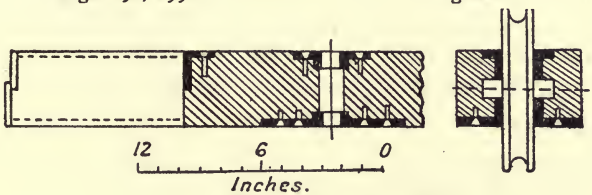


Fig. 200.



Figs. 201, 202.

rope that carries the load, and is held by a man on the ground, and prevents the load touching the tower and disfiguring the masonry. The slings may be of rope or chain ; if of chain they should have a large ring (say about 3 ins. diameter) on the single ends, the chain being double, and long enough to take in the required number of bricks. The pails are best made of wood, rather long, and of dairy pattern, with lugs on each side through which a strong rope passes, and knotted at the ends.

The man at the tower bottom who hooks on materials, etc., must be protected by a lean-to wooden shelter like a roof extending from the side of the tower opposite the guide

pulley to near the hoisting rope ; or a small bogie may be used. Two lengths of rails (about 6 ft. long) are laid at right angles to the tower, the centre being the heaving rope, and on these rails the bogie is placed, which has both wheels near the centre of the frame. The lime or the brick in the slings is built on this bogie at the outmost end of the rails ; the bogie is then run in against the tower, and when the material is hoisted the bogie is drawn out again. This keeps the man out of the way of falling material.

Masonry Pillars.—When the height of a masonry pillar greatly exceeds its dimensions on plan the provision of sufficient cross-section to take the load within a given safe stress on the material, stated in Chapter I, is insufficient. Such cases are rare, because in buildings the pillars are nearly always of steel, but in the case of a very high masonry viaduct it may be necessary to consider the case from this point of view.

Crushing or breaking by compression is not a simple phenomenon like tearing, but is more or less complex and varied, according to the nature of the substance. If f= resistance of material to crushing, which in the case of stone may be taken as follows in lbs. per square inch :—

Granite	5500 to 11,000
Limestone	4000 to 4500
Sandstone (weak) . . .	2200
,, (ordinary) . .	3300 to 4400
,, (strong) . . .	5400

and A the cross-sectional area and P the crushing load, obviously

$$P=fA \quad . \quad . \quad . \quad . \quad (141)$$

provided the load is centrally applied. If the load is eccentrically applied and x_0 is the greatest deviation of the centre of pressure from the centre of figure in any cross-section, that is the greatest deviation from the line of axis of the load from the axis of the pillar, and x_1=distance of point of greatest stress from the axis of the pillar.

Then, if the moment of inertia

$$=I=\int x_0^2 y dx_1$$

the crushing load is

$$P=\frac{fA}{1+\frac{x_0x_1A}{I}} \quad . \quad . \quad . \quad (142)$$

In the case of a masonry pillar which is rectangular, square, or circular, the value of $\frac{x_1A}{I}$, (if the neutral axis is the diameter to which x_0 is perpendicular,) is equal to :

1. Rectangle db (b=neutral axis) $\frac{6}{d}$
2. Square $d^2=\frac{6}{d}$ . . . (143)

3. Square (hollow) d_1 and b_1
being inside dimensions and d and b outside then

$$\frac{x_1A}{I}=\frac{6d}{d^2-d_1^2} \quad . \quad . \quad . \quad (144)$$

and a hollow circular section d and d_1 being inside and outside diameters

$$\frac{x_1A}{I}=\frac{8d}{d^2-d_1^2} \quad . \quad . \quad . \quad (145)$$

Now P should never be less than $\frac{fA}{2}$ $\therefore x_0$ should never be less than $\frac{I}{x_1A}$ if this condition is to be fulfilled.

Now most kinds of stone, when crushed to destruction, will shear or slide along the line xy in Fig. 203.

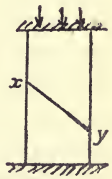


Fig. 203.

Now let f be as in previous table, and c a constant for masonry pillars, and considered as fixed at both ends$=\frac{1}{600}$. Then the safe load P on any pillar of length$=l$ inches and d=least external diameter in inches.

$$P=\frac{fA}{1+c\frac{l^2}{d^2}} \quad . \quad . \quad . \quad . \quad (146)$$

In other words, in long pillars or piers of masonry loaded on

top, the value of P in Equation 141 no longer holds good, but that of 146, which will be less than the former value, must be used and A increased in consequence. The author does not know of any tests on actual masonry columns of any size, but some full-size brick and terra-cotta block columns were recently tested to destruction at the Illinois University by Messrs. Arthur N. Talbot and D. H. Abrams and communicated in a Bulletin, No. 27, dated 29th September, 1908, and the conclusions the experiments gave are what the author as a practical man would expect and are here quoted :

" Both brick columns and terra-cotta block columns gave high strengths in all cases where strong mortar and care in building were used. For central loading, the strength of the brick columns ranged from 3220 to 4110 lbs. per square inch, and the strength of the terra-cotta block columns from 2700 to 3790 lbs. per square inch, the columns having the highest resistance not failing at the full capacity of the machine. The effect of the strength of the mortar is apparent in the carrying capacity developed in the columns ; lower loads were found in columns built with 1 to 5 Portland cement mortar than in those with 1 to 3 Portland cement mortar, still lower loads in those with 1 to 3 natural cement mortar, and still lower loads in those having 1 to 2 lime mortar. The effect of the quality of the brick is shown in the columns made with inferior brick, which carried only 31 per cent as much as columns built with the better grade of brick. In the case of the terra-cotta columns, the blocks which were culled out as somewhat inferior gave a column strength perhaps 30 per cent less than the columns built with superior blocks. The effect of the attempt to represent hurried or careless workmanship in two brick columns and in three terra-cotta block columns was a loss in strength of about 15 per cent and 25 per cent, respectively.

" The ratio of the strength of the columns to the compressive strength of the individual brick and block is of interest. In the well-built brick columns loaded centrally, the ratio of strength of column to compressive strength of individual brick ranged from 0·31 to 0·37, and in the

underburned clay brick column the ratio was 0·27. In the terra-cotta block columns with central loading the ratio of strength of column to that of individual block was 0·74 for the incompleted test and 0·83, 0·85, and 0·89 for the others. If, as seems to be the case, the strength of the brick or block to resist cross-breaking is an element in determining the strength of the built-up column, a deeper or thicker brick would give higher column strength. It is possible that this partially accounts for the fact that the ratio is found to be higher for terra-cotta block columns than for brick columns. The tests suggest that the ability of individual pieces to resist transverse strength is an important element in the strength of the completed column. This suggestion may have an important bearing on the advantageous size of the component blocks which may be used in a compression piece where high strength is desired.

" The strength of the column is greater than the strength of the mortar cubes in both brick and terra-cotta block columns, excepting only the soft brick columns which had brick of low compressive strength. It is evident that the strength of individual brick or blocks and the strength of the mortar both enter into the resistance of the column. The relative effect of the two depends upon the character of the material. It is evident, however, that the better the individual piece the more important it is to have a mortar of high resisting strength.

" The results obtained in applying the load eccentrically were found to agree very well with those obtained from ordinary analysis. When the amount of eccentricity in the application of the load is known or may be estimated closely, the ability of the column to resist this action may be calculated quite closely. It is apparent from the results that the calculated resisting stress in the column on the side of maximum compression is higher than that which causes failure in centrally loaded columns. The higher stress developed with eccentric loading is probably due to the influence of the restraint of the less stressed interior portion. The tests made by applying and releasing a single load a number of times gave failures at loads below those

which produced failure in similar columns at a single application of the load. The phenomenon is common in materials of the nature of brick and terra-cotta.

"It is apparent that the quality of workmanship in laying up such columns has an important bearing upon the resisting strength. The work of building columns, however, is not difficult and requires only ordinary care. Full joints and an even bearing are important, and the ordinary workman ought to be able to construct columns of high strength."

CHAPTER VIII

MONOLITHIC AND BLOCK CONCRETE CONSTRUCTION

NOWADAYS so important a part does concrete play in the construction of what may be broadly termed masonry work that a book dealing with the subject would not be complete without some reference to it. This has already been pointed out in what has been said in reference to works built partly of concrete blocks in place of stones. As the heading of the chapter implies, the following remarks are confined to monolithic construction, that is, works constructed of concrete as a whole, which may or may not be faced and otherwise made complete with stone and concrete blocks, used as a substitute for stone blocks. This latter method may be employed where monolithic work is either undesirable or impossible, and where stone is either scarce or not obtainable in blocks of sufficient size. No reference will be made to the modern methods of reinforced concrete or ferro-concrete, as it is called. This is a special study, and is treated in a large number of text-books and in the author's *Civil Engineering Practice.* It is hardly necessary to state that Portland cement is the most important ingredient of concrete. Its manufacture is not a subject for discussion here. It is sufficient to say that it is a market product, and one which has reached a high degree of perfection. It is an important industry in England and, without prejudice to other countries, that produced in England is far superior to any other. For information on testing cement the reader is referred to other sources of information again, and it only remains to say that the cement should come up to the standards required by the

standard specification* of the British Engineering Standards Committee, and that the cement should only be obtained from makers who have a reputation to maintain, and should always be *artificial* cement. It is necessary to draw attention to this term, as there are some *natural* Portland cements imported from Belgium and elsewhere, and they are without exception inferior to artificial cement. So much for the cement which, when obtained as advised and under the specification stated, may be looked upon as a reliable product which the engineer will test from time to time. By far the most important point so far as the reader is concerned is the selection of a suitable aggregate. This may often be limited to materials found near the site of the works. By far the best is broken stone, but the stone must be selected with care. Hard limestone, if free from dust, makes very good concrete, but dust in any aggregate is detrimental to the setting of the concrete. Crushed granite is also a very desirable material, but is liable to dust and must be well washed. It must not be inferred because granite is a very strong stone that the resulting concrete will be extra strong. This does not follow at all. Under the heading of granite may be included any hard volcanic rock such as basalt, quartzite, trap, and whinstone. Crystalline rocks such as diorites, which fall into a class between granite and lime or sandstone, are generally good aggregate, but sandstone does not form a strong concrete, but may be safely used in heavy works if well washed. As a rule broken stone is used in works of the kind under consideration.

Ballast is the term applied to heavy gravel or shingle. It may be had from rivers, sea-shore, or pits, the latter very often having a large proportion of loam, which under all circumstances must be washed away, because its presence on each individual stone prevents proper adhesion of the cement to that stone and weakness of the mass will result. It should not be used uncrushed, not because the stones are too large, usually they are about the right size for heavy work, but because the smooth surface of the

* Published by Crosby, Lockwood and Co.

stone is detrimental to thorough bonding of the mass. It must be screened and graded in size before use.

Broken brick is an excellent material, as, being slightly porous, it gives an increased tenacity. It must be well washed to remove dust, which will be produced in abnormal quantities if the bricks are bad or they are crushed too much. The engineer should, however, always inspect any old bricks proposed to be used for this purpose and satisfy himself that there is no old mortar and plaster on them, which is very detrimental to concrete by acting chemically on the cement and causing it to blow and crack.

Burnt clay ballast may be said to be a last resource when other material is hard to get and the works are near a clay field. The clay may be burnt, and if well burnt can be used for concrete, but is not very desirable, although in large blocks in which weight rather than strength is the dominant factor there is nothing much against it.

Trass and pumice, when found near the works, are useful for the aggregate. The former was used with good results by Mr. Frank Latham on sea-wall works at Penzance.*

Slag and breeze are used in fine work, but are usually too light for heavy engineering work.

The aggregate must be broken to proper size. It may be done by hand or machine, those with a knapping motion (such as Baxter's) are by far the best. It should be screened by a rotary screen attached to the machine. In all cases the aggregate should be washed, and this on large jobs is done by a machine washer also.

Sand is essential to good concrete. The duty of the sand is to fill up the spaces between the stones of the aggregate, the cement finally filling in the remaining voids and binding the whole together. It should be selected with great care, and coarse sand is in all cases preferable to fine or dusty sand. It must under all circumstances be clean, and if not clean it must be washed. A mixture of fine (not dust) sand and coarse is the best, as the volume of voids is reduced. It must be borne in mind by the reader that by reducing the voids in the mixture before the cement is

* *Trans. Civil Engineers in Ireland*, Vol. XXXII.

added the quantity of cement for a given mixture may be reduced without reducing the strength. This is not always appreciated as well as it might be, and consequently cement is wasted. Pit sand may be used, but river sand is much better. Sea sand may also be used, and for heavy work has no objections when washed, as the presence of salt does not affect Portland cement, although it retards the setting, and for this reason salt water MAY be used for mixing if fresh water is scarce.

Mixing.—Concrete should be proportional and mixed in such a way as to afford the greatest possible strength with the least amount of cement, and is simply a matter of care and forethought on the part of the engineer. The whole thing is to obtain maximum density by reducing the voids to a minimum. A strong mixture does not mean a strong concrete. When the remaining voids are filled with cement any excess of cement is waste. To secure a perfect concrete the aggregate must be so proportioned and graded in size that the percentage of voids is reduced to a minimum. There should be sufficient smaller stones to pack between the spaces of the larger ones and then still smaller stones to pack between these, and finally sand to fill those remaining. The final duty of the cement is to coat every particle of the aggregate and bind it to its neighbours. In heavy work large stones are often used in the concrete. They are called " displacers " or " plums " and are placed by hand, and in such a way that they do not touch each other. When stone is plentiful they will reduce the cost of the work without any bad results. Angular stones for the concrete are preferable to round ones, and sharp sand to smooth. Various grades of aggregate should be used in the same mixture, varying from $2\frac{1}{2}$ ins. to $\frac{1}{2}$ in. Below $\frac{1}{4}$ in. the small particles are considered as sand. Siliceous sands are in all cases the best. The proportions used for concrete cannot be laid down with any strictness. Various proportions have been used. In the author's opinion, for heavy works engineers have favoured too much cement, and been content with too little mixing and unsystematic proportioning. For heavy concrete not subject to much load, however,

a 1 to 4 to 8 mixture of cement, sand, and aggregate may safely be used, if the last-named are carefully selected and well machine mixed. For most other work, such as bridge piers and abutments and retaining-walls, a 1 to 3 to 6 mixture should be used. These, of course, are merely suggestions. The best mixture for any concrete can be determined experimentally as follows :—

Procure a piece of iron tube and make a mixture similar to that which it is proposed to use. Very carefully note down its composition, then mix it and ram it into the piece of tube, which was previously sealed at one end by means of a screwed cap. Carefully measure its height in the tube when fully rammed. Then clean out the tube again and make a fresh mixture, varying the *proportions* of sand and broken stone, but still having the same total weight of the two combined. If less or more, it will be a guide to the future adopted proportions. Again, note the height of the sample when rammed. Repeat this process a few times. The *lowest* height proves the *best* mixture, but they must be very accurately proportioned if they are to be of any practical value.

The aggregate will, however, shrink during mixing. For example, say we have $4\frac{1}{2}$ cub. ft. of Portland cement, 9 of sand, and $20\frac{1}{2}$ of aggregate. The resulting mass, which it might be supposed would yield 34 cub. ft. of concrete, will only yield in practice about 21·4 cub. ft. in mass.

The use of salt water in concrete works has the tendency of retarding the setting power. All water used should be carefully selected ; also it must be clean, free from acid and strong alkalies. It must not have much lime present, and must be free from organic impurities. The average quantity required would be from 21 to 24 gals. per cubic yard of dry mixture.

Concrete should not be tipped from a height, as might be considered an advantage, nor should it be applied in frosty weather. If, however, this would be quite unavoidable, as in the case of cold climates, 1 per cent of salt may be added with advantage, or a warm mixture used. The average composition of a cubic yard of concrete would be as

follows, according to the proportions used to make 1 cub. yd. of concrete :—

TABLE VIII

Proportion.	P. Cement in lbs.	Cub. Yd. of Sand.	Cub. Yd. of Ballast or Stone.
1–2–4	590	·44	·88
1–2½–5	485	·45	·91
1–3–6	414	·46	·93
1–4–8	320	·48	·96

On works of any size or importance the concrete is mixed by machinery, Ransome's and Taylor's rotary mixers being the best known. They need no description, as they are put up ready for work and are driven by a small steam-engine. In the case of hand mixing attention should be paid to the way it is done. A wooden platform is erected for the purpose, and boxes without top or bottom used for measuring. They should be made 1 ft. deep and of such a size that, when one is filled with aggregate levelled off on top and the others with sand, they contain the correct quantity of these materials for one sack of cement. The whole is then turned over twice dry and watered through a *rose only* (on no account must water be poured on by a bucket). The condition of the concrete should be fairly stiff, too much water washes away the cement. The machine mixers referred to mix one batch of concrete and then require reloading. They are preferable to continuous mixers. Concrete when set will usually have a number of fine cracks. These need not alarm the engineer, as they are usually superficial, but any suspicious ones should be looked to and their depth ascertained with a penknife. They are due to the thin coating of cement which always finds its way to the surface of concrete work.

Placing Concrete.—The placing of concrete is a subject which cannot adequately be described on paper. In engineering work the circumstances of each case differ so much. For the *forms* timber is nearly always used, except in the case of steel piling designed for the purpose and

before described, and culverts. The timber used should be quite cheap, as it is not much use afterwards. It must be remembered that concrete, being in a semi-fluid state, exerts a pressure on the forms, which must be of thick timber (1½ and 2-in. deals are generally used), must be well stayed and strutted, and should have tongued and grooved joints. In the case of large walls they are tied from side to side with bolts to prevent any bulging. The bolts are well greased to admit of removal; in the same way the boards are coated with soft soap. It is important that the jointing should be properly done, as bad joints leave a very ugly surface on the concrete, and in engineering work the surface so left is generally that which remains, plastering being expensive and always unsatisfactory. Nor should damaged timber be used for the same reason. No concrete should be placed till the forms have been seen by a responsible person, who should test them vertically and the efficiency of the strutting. The timber used should be spruce, not too well seasoned, otherwise it will swell when wet and warp, and the joints will open and the concrete fall out.

One of the most important things in connection with concrete work is to leave the forms in place till the whole mass has set thoroughly. There are no rules for time and no definite results to guide the designer as to how long concrete continues to harden. In the case of heavy works this continues for a very long time; in fact, long after the forms may be safely removed. The writer is of the opinion that in heavy work it may go on almost indefinitely. One month is not any too long to leave forms in place where the time can be spared, but this is not always practicable, and if the weather is warm, not essential, but frost and damp weather will delay and even retard setting altogether for a time. When putting concrete into place the mixture should not be too wet or too dry. It should be of a consistency for placing in layers of 8 ins. or 10 ins. and tamping lightly with an iron rammer until the water oozes up to the surface and none of the stones are left uncovered with cement. It is a great mistake to tip concrete from a height, which may

at first be thought to consolidate it, because by so doing the larger materials become separated from the smaller ones and air is likely to get into the work and cause it to be honeycombed. In large works, as before described, concrete often has to be placed in mass under water. Although the use of large blocks set in air is preferable, yet it is not always practicable, and concrete can be placed under water in mass without any evil results. It may be done by means of a long tube, and if carefully done the resulting concrete will set hard under pressure of the water to which it will be subject, and concrete set under water, if carefully placed, is very homogeneous, and when set, which is a slow process, is very dense and strong. If placed in water the surface of which is not quite still, however, the results will be unreliable. For general use in such cases special tipping buckets are used. That known as Ransome's is made so that it is slung on to a crane hook from the four corners and one of the sides is a lifting door. When down at the point where it is required to place the concrete this door is raised out of its grooves by a chain and at the same time the bucket, which is oblong in shape, tilts downwards at the door end and the concrete is deposited with the least possible disturbance. The Ransome ver Mehr Machinery Co. of Caxton House, London, S.W., are makers of all kinds of concrete-handling machinery. For depositing concrete in rough water these buckets have top-covering doors.

Waterproofing Concrete

In the case of retaining-walls for water, concrete is often used in place of masonry. In large dams the thickness of masonry is usually enough to secure a watertight structure; at the same time all masonry dams do admit of a very small amount of water getting through them somehow. Contrary to public opinion, concrete, if mixed with great care to give a very dense mixture, is waterproof without any further treatment, at least, enough for any practical purpose. In the case of tanks, however, where the concrete is comparatively thin, some means of rendering the same quite watertight is generally resorted to. This may consist of

internal plastering with neat cement or 1 to 1 cement and sand, but however well done this plaster is liable to fail after a time, and the author is inclined to favour asphalte as the best possible medium. This class of work should be entrusted to firms who specialize in it and will guarantee their work, as the best (and cheapest in the end) results will obtain. This waterproofing should be laid 6 ins. or 9 ins. below the finished surface of the work which will be in contact with the water. Waterproofing compounds for mixing with concrete (such as a 5 per cent solution of alum in the water used for mixing) are of doubtful utility and reduce the strength and delay the setting power of the concrete. A little lime putty, however (2½ per cent), makes the concrete denser and easier to work, without any reduction of the strength. In the case of sea-water the density of the mixture should be most carefully attended to, as although sea-water has no evil effects on the concrete as a whole, if it is not dense the water will penetrate it, and sea-water appears to have an evil effect on PARTIALLY set concrete, the sulphate in solution reacting with the lime in the cement and forming calcium sulphate, which will result in gradual breaking up of the mass.

In the case of concrete deposited under water a quick-setting cement is desirable, and such cements, specially made for this purpose, are obtainable from the leading makers. No concrete is *absolutely* impervious to water, but it can be made very nearly so, and if sufficiently thick very little water will penetrate even under considerable pressure, and even if it does it appears to stop after a time, possibly by the pores (through which the water finds innumerable minute passages) getting choked up.

There is one great danger in impervious concrete, and that is frost, which will successfully destroy the largest structures if allowed to attack them when in a state of semi-solidity, and no important concrete work should be done in frosty weather, and any likelihood of night frost must be most carefully guarded against.

For all kinds of heavy work concrete has an infinite variety of uses for the engineer. In the first place piles,

which are often essential to masonry construction, are now made of reinforced concrete. They are equal in most other respects to timber piles, although more expensive, but have the great advantage of resisting aquatic parasites, which destroy timber piles, slowly but surely. They are usually reinforced, but *may* be made without. In the former system a regular skeleton of steel rods and bars are set up attached to a steel shoe and round which concrete is placed in a mould. This work comes strictly under the heading of reinforced concrete, and the process is described in books referred to before. In the case of piles of this description they are driven in precisely the same way as a timber pile; the cap, however, contains a sand cushion to prevent damage to the pile. The piles may be made with a hole in the centre, down which a water-jet may be forced, which will assist the driving of the pile; where no reinforcement is used a steel tube well greased inside is driven into the ground and concrete poured into it. The tube is subsequently withdrawn. For moderate depths the system is advantageous, but at great depths the tubes are very difficult to withdraw. Cast-iron cylinders for foundations are filled or partially filled with concrete. When partly filled a form of the type shown in Figs. 204 and 205 is used, the latter figure showing how the frame battens are cut. The value of C is equal to

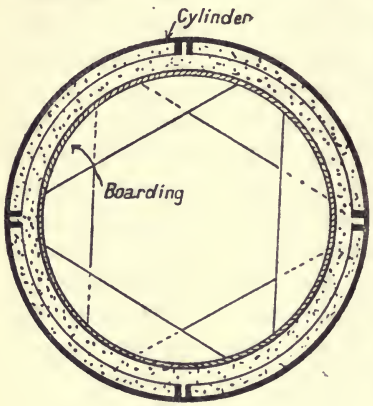


Fig. 204.

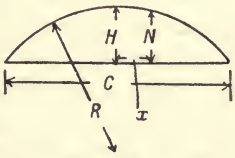


Fig. 205.

$$C = 2\sqrt{R^2-(R-H)^2} \quad . \quad . \quad . \quad (147)$$

$$H = R-\sqrt{R^2-\frac{C^2}{4}} \quad . \quad . \quad . \quad (148)$$

$$N = \sqrt{R^2-X^2}-(R-H) \quad . \quad . \quad (149)$$

Caissons, before described, are now made of concrete in place of steel, and often of large size. For harbour and bridge work they are very useful. The bottom section would have a cutting-edge of cast iron, and upon it would be built a section 4 or 5 ft. deep, of which a typical form for a bridge pier would be that shown in Fig. 205*a*. The weight of each section could be made up to about 10 tons, or more if they could be conveniently handled. The joints would be tongues and grooves, the tongues being wedge shape. In the grooves cement would be placed, which would be forced up on lowering the sections into one another. The lowering would have to be very carefully done, and a system of steel guide-rods placed in the holes shown of the bottom section and long enough to pass through

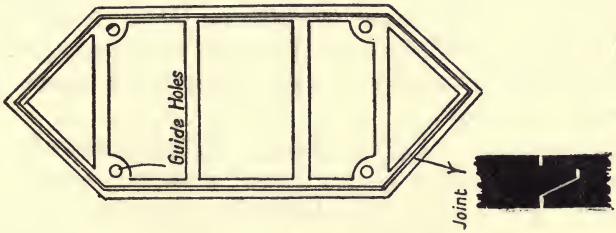


Fig. 205*a*.

the holes of each upper section would be provided to ensure accurate work. The sections, of course, would have to be most accurately moulded, steel forms being preferable to timber ones for this purpose. The sections should be allowed to harden for at least twenty-eight days. This class of work was used on the Manchester Ship Canal by Mr. W. H. Hunter, M.I.C.E., and on the Foyle Bridge of the Letterkenny and Strabane Railway (Ireland), and quay walls at Ymuiden, Holland.

The use of concrete when reinforced with steel is almost infinite for constructing engineering works which were formerly constructed of solid masonry. For heavy work the use of concrete blocks is fairly obvious, but the variety of circumstances attending its use in the hands of the engineer makes it almost impossible of treatment on paper. The

blocks used are quite simple to make, the provision of lifting appliances in the block, either permanent or removable, being the principal thing. Bridges, if small, may be made of simple concrete, but reinforced is much more economical. For arches the centering is precisely the same as for any masonry arch, except that the laggins must be close and the sides boarded and strutted to retain the liquid mass. It must also be borne in mind that the load of this liquid mass may be greater than the stones of a masonry arch, and the centres must be extra strong accordingly.

Culverts.—A few words may be said on the subject of culverts, which may be economically built of concrete by means of steel collapsible centres, as made by the Blaw Collapsible Centering Co., Dock House, Billiter Street, London, E.C.; such a culvert was made recently at Blyth, Northumberland, for the North-Eastern Railway Company, of which the following description is taken from the author's *Main Drainage of Towns:*

"The culvert is of 3 ft. internal diameter, 1800 ft. long, formed of two rings of brickwork and surrounded with concrete. The culvert is included in a contract for certain bridges, etc., let by the North-Eastern Railway Company to Mr. T. Telford Gibb, contractor, of Westminster, and will be handed over to the Blyth Urban District Council on completion.

"Steel centres in segments 5 ft. in length were used for forming the concrete invert 4 ft. 6 ins. in diameter, and for the top half of the brickwork centres, 3 ft. in diameter; the centering was set in lengths of 50 ft. at a time.

"The excavation being down to the full depth, 9 ins. of concrete was deposited in the trench, bringing it up to the level of the bottom of the invert; then the 4 ft. 6 in. centres were placed on the top of this, being held in position and prevented from rising by timber placed at intervals over the top end wedged against the sides of the trench: the concrete was then rammed in round the centres and allowed to set, and the centres then removed. The brickwork for the bottom half of the culvert was then built, and on the top of

this the 3-ft. diameter centres were placed and the upper half of the culvert brickwork built on these. Then the remaining concrete was formed on the top of the brickwork.

"The steel centres were withdrawn by tightening the union nuts, which withdrew the centres from the formed brickwork or concrete, and union nuts were then loosened, and the rods and union nuts removed, and the centres were drawn forward, then the rods were replaced and tightened up, and the centres were then ready for further use.

"The centres made a very smooth and perfect surface and form on the concrete and joints of the brickwork, and, excepting brushing, did not need to be cleaned or repaired in any way.

Similar steel centres may be used almost of any size for bridge work, in fact they were advised in a previous chapter, and a suitable form is shown in Fig. 206. For walls and kerbstones of footpaths and all places where small blocks of stone are required, and where they do not support any important or severe load, small concrete blocks may be substituted for stones. They hardly come into the province of the engineer, but rather the builder and architect. They are made by machinery in special block-making machines, and for the purpose they are useful, for they are undoubtedly cheap. For facing work which is concrete in the mass and in which the facing is for appearance only they are very useful in temperate climates, as with such machines any type of face can be made, the quarry-face with drafted margin being very pleasing. Such blocks, however, are quite unsuitable for taking any load or strain.

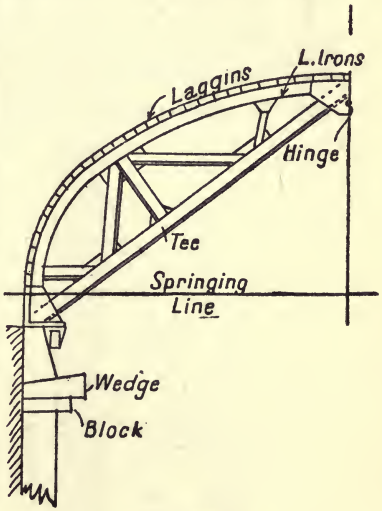


Fig. 206.

CHAPTER IX

SHORING AND UNDERPINNING

THE work of the engineer consists in converting the sources of nature to the use and convenience of man, but oft-times nature steps in and spoils his handiwork. Moreover, the engineer like other men is liable to make mistakes, and in consequence we find that structures fail. Except in rare cases they do not fail without timely warning,* and to ensure the safety of the structure and the public the art of shoring and underpinning (in other words propping it up) is resorted to. The most commonly occurring case is that of a wall getting out of the perpendicular and being liable to fall. When the ground on the off side of the wall is firm a raking shore can be put up till such time as the wall is secured or taken down. A simple form of raking shore is shown in Fig. 207. It is composed of the wall-piece *a* usually 9 ins. × 3 ins., long enough to secure the main part of the failing wall. Into this wall-piece is cut a rectangular hole, in which is inserted a piece of wood *b* which is called a needle and sometimes a "joggle" or jossle. It serves as an

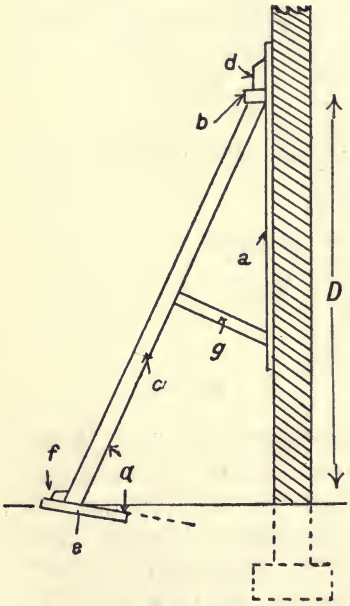


Fig. 207.

* In the autumn of 1913 a building *did* fail suddenly and without warning in Dublin, unfortunately with fatal results.

abutment for the head of the shore *c*; above it there is a cleat *d* nailed to the wall-piece just above the needle, and which prevents it being forced out of place. This shore at the bottom rests on a balk of timber *e*, which is called a sole-plate, upon which is another cleat *f* to prevent it slipping, *g* being known as a secondary strut. This constitutes the simplest form of raking shore. It will be seen that the sole-piece is not at right angles to the strut, but that the angle *a* is less than 90°. It is usually about 10° or 15° less,

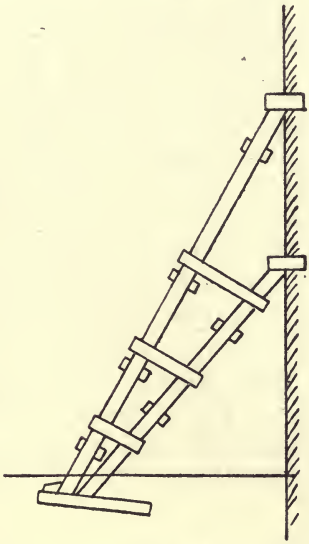


Fig. 208.

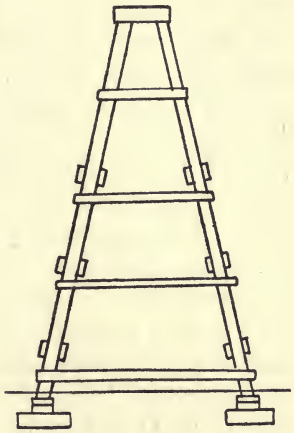


Fig. 209.

and this is to ensure that it shall be as nearly as possible at right angles to the resultant of the load due to the weight of the wall and that tending to overturn the wall, and supposed to be concentrated at *b*. By levering the shore along the sole-piece it is tightened up. It must be gently worked, hammering wedges is liable to set up vibration and aid rather than prevent failure. A groove is cut at the bottom of the shore immediately above *e*, by means of which a crow-bar can be inserted for the purpose of tightening up. The duty of the small strut *g* is to take up the cross strain, always acting perpendicular to the shore.

The above example may be amplified into a shore containing up to four or even five struts. Each strut is treated as a single one. An example is shown in Figs. 208, 209. The wall-piece is, of course, longer than before, and has four holes in place of one, and several secondary struts in sets are used which are usually 7-in. × 1-in. timbers. They are nailed to the side of the wall-piece. When the struts are of too great a length to be had in one piece two must be used, and tightened by oak wedges. The strut above these is called a *rider;* 5 ins. × 5 ins. is a usual scantling for this work, and the shores are usually placed 12 or 15 ft. apart. When tightened up the members are secured by iron hooks of ⊔ shape, pointed at the ends, which are driven into the timbers. They can easily be removed. A cleat, of course, as shown at *f* (Fig. 207) is always necessary against the foot of the outer shore. When more than one strut is used they either butt against each other or are tightened to each other by small blocks of wood. Solid ground is, of course, essential to stability, and when it is not solid, as may be the case when shoring up a river wall, piles must either be driven or a large cast-iron plate provided as a sole-piece, whose area is great enough to give a safe intensity of stress. For instance, let W= weight of wall in cwts. and *t* thickness of wall in feet at the ground-line and D as shown in Fig. 207. Then the horizontal thrust Q at *b* is equal to

$$Q=\frac{Wt}{2D} \quad . \quad . \quad . \quad . \quad (150)$$

and if α is as shown the vertical force acting down the centre of the wall is equal to

$$P=Q \tan \alpha-\frac{W}{2} \quad . \quad . \quad . \quad (151)$$

W being the weight of the shore in cwts. Then the load on the shore, which has to be supported by the ground, is equal to

$$R=P \sin \alpha+Q \cos \alpha \quad . \quad . \quad (152)$$

If any shore is of dimensions L and *d*, L being length in feet and *d* width in inches, the safe load on the strut, which

should be about 1·5 R, is equal to

$$c\frac{d^4}{L^2} \quad . \quad . \quad . \quad . \quad (153)$$

c being a constant=15·5. In the case of retaining-walls a somewhat different kind of strut is more useful than those shown. It is illustrated in Fig. 210, and it should be braced all ways. In the case of a cutting in which one wall has failed and the traffic will not be interfered with this wall may be supported by struts to the other, in the way shown in Fig. 211, in which there is one horizontal strut *a* made tight with oak wedges. It is also stiffened with raking struts *b*, which butt against cleats on the wall-pieces *c* and straining-pieces *d*. This class of work is much used for propping up houses, but is usually inadvisable in engineering work, except in very rare cases. There are, of course, various ways of shoring up walls, and the engineer has often to consider for a long time how to deal with any particular case. The examples given are merely typical and suggestive.

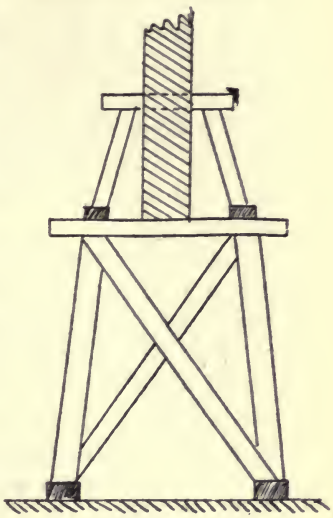


Fig. 210.

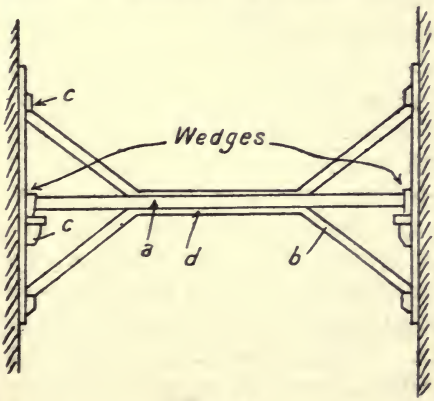


Fig. 211.

Underpinning. — We now have to consider what is to be done when the work is temporarily secured to ensure that it will remain secure after the shoring is removed.

In the first place the engineer should satisfy himself that the work can be repaired without rebuilding. A wall failing by

subsidence can usually be underpinned, but if cracked and bulged by pressure it may be almost futile to try and save it. In rare cases it may be possible to tie it back by plates on the face and land ties, which have been previously discussed. In this case the land ties will have to be most securely fixed, because to retain the wall in its condition of collapse is not enough, they must withstand the drawing up of the wall to its normal position again. This is done by heating the tied rods, which will have to be of very stout section, and screwing up the nuts while the rods are hot, on cooling the wall will be gradually drawn up into place.

Failure of walls and works in general by sinking is a common occurrence and can usually be remedied. Walls which merely show signs of sinking may have small sections of their foundations cut away and a firm bottom exposed which is built up tight to the wall. In the case of river and canal walls a cofferdam will probably be required. Concrete in bags is useful for the purpose. Where the foundations of a wall have been scoured away a toe projecting 2 or 3 ft. and consisting of concrete or rubble in cement placed behind sheet-piling is a useful expedient. In many cases of failure, however, it will be necessary to raise the wall back into its original position, which is often a slow and difficult procedure; it means opening up the ground in sections under the wall, which in the case of water means a cofferdam. If a hard bottom can be reached the work may be raised in sections by hydraulic jacks and packed underneath with masonry or concrete. Failing this some form of artificial foundation will be necessary. If both sides of the wall can be got at, ordinary piles may be driven close to the wall on each side and the space between them immediately under the wall bridged across by timbers and cross-timbers, and the whole filled up to the level of these timbers with concrete, upon which the work may be carried up to meet the old work. Where the back of the wall cannot be got at driving piles on one side only is obviously useless, and to drive them under the wall impossible. There remains one expedient, that of screw piles. Even this is a

difficult job, owing to the confined working space, but has been carried out with success by the writer. A similar system, only in this case there were two sets of piles, was used for underpinning the town hall at Yarmouth. See Fig. 212.*

The prominent features of this scheme were :

1st. The insertion of cast-iron screw-piles at intervals of about 9 ft. inside and outside the main walls. These piles were about 23 ft. long, screwed 3 ft. into the gravel bank, the metal being 1¼ in. thick, and the screw 12 ins.

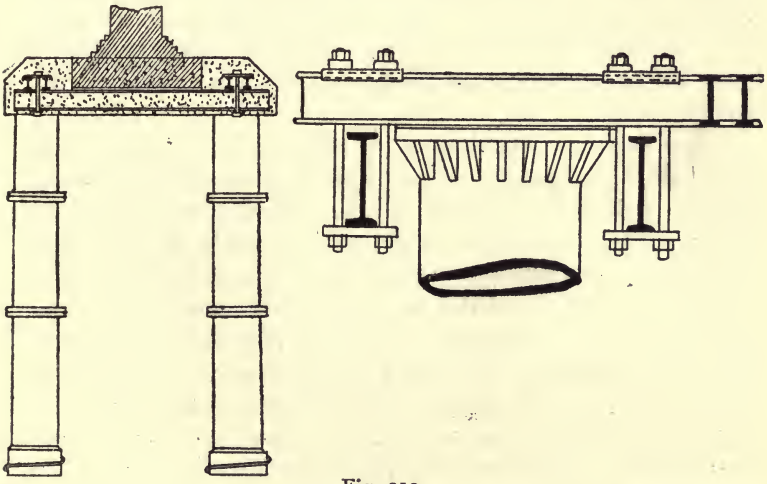


Fig. 212.

greater in diameter than the piles and of 6 ins. pitch, the piles when down were cleared of earth and filled with cement concrete. The diameters of the piles were 2 ft. and 2 ft. 6 ins., to carry an approximate weight of, say, 35 to 50 tons, respectively ; but it being stated that the soil of the district injuriously affected cast iron, the diameters were increased to 2 ft. 6 ins. and 3 ft., that, in the event of the iron failing, the internal concrete columns might safely take the weight.

2nd. Double lines of rolled girders were placed on top of

* *Trans. I.C.E.*, Vol. XCVIII.

the piles, with a $2\frac{1}{8}$-in. space between them and parallel with the walls.

3rd. Wrought-iron needles 14 ft. to 16 ft. long, consisting mostly of rolled joists, 16 ins. by 6 ins., with a top table added, of a length equal to the width of the original concrete and 12 ins. wide, passed through under this concrete foundation at intervals of about 3 ft., were suspended at each end by two 2-in. bolts from the beforementioned girders. The connections were made by having a $\frac{3}{8}$-inch plate 15 ins. square placed on top of the girders turned down to clip them, and having two 2-in. clearing holes punched therein. Two 7-in. wrought-iron washers $1\frac{1}{2}$ in. thick on this plate, and one spectacle-piece 14 ins. by 6 ins. by $1\frac{1}{2}$ in. under the needle or cross-girder. The bolts had 12 ins. of thread.

Upon such a foundation, of course, the raising and subsequent pinning-up of the wall is comparatively easy. Where, owing to confined space, screw piles cannot be driven plumb under the wall and a double row of piles impossible there may be had resort to iron cylinders. They are driven as close to the wall as possible, and from them and under the wall project stout and very stiff cantilevers. When one considers the action of the load on a cantilever the one thing which appears difficult is to hold down the cantilever to the cylinder in such a way that overturning is impossible. This may be efficiently done by casting the cantilever to the top section of the cylinder. It is hardly necessary to add that the expense of this form of underpinning prevents it being used except to save the most important works, such as bridge piers, etc.

In the case of a masonry wall in which part or whole of the bottom is to be cut away, needling is resorted to. It consists in cutting holes in the wall about 14 ins. square at intervals of 5 to 7 ft.* Through these holes are inserted short, stout pieces of timber usually 12×12. These in turn are propped up on either side by stout posts bearing on the ground through the medium of sole-pieces, and

* Masonry in most cases will stand unsupported over this small space except when badly cracked.

tightened in the same way as raking shores, so that the needles press tightly upwards against the masonry it is desired to support. Raking shores are usually fixed as an additional security, but if the wall is in good condition and the removal of a part of it is for alteration rather than repair this may not be necessary. The scantling of the timber may be found as follows: If we know L=length of bearing, W=weight of wall in lbs., c=constant=·011 for fir and ·014 for oak. Then if b and d are dimensions of timber in inches

$$d=\sqrt[3]{\frac{L^2\times W\times c}{b}} \quad . \quad . \quad . \quad (154)$$

or alternately

$$b=\frac{L^2\times W\times c}{d^3} \quad . \quad . \quad . \quad (155)$$

When the structure is propped up preparatory to putting in new foundations or underpinning, the whole thing is to pack in the concrete or masonry so that no chance of after settlement can possibly take place. There are various ways of doing this. In the case of rubble, it may be built up bit by bit with small stones and strong cement mortar packed as tight as the mason can possibly do it. Bags of concrete may be similarly used, the closing up being done with small bags of sand and cement concrete without any stones when the top load is not heavy. Either of these methods is satisfactory. Concrete will expand when set, and this is an advantage in underpinning work; in fact, many engineers prefer lime concrete only for this class of work due to the extra expansion. On the other hand, this expansion will tend to move laterally, which it must be prevented from doing by timbering. In the case of a wall in which both sides may be got at two rows of timbers tied to each other by bolts will serve the purpose, and the screwing up of the bolts, which should be 1 in. or 1½ in., and spaced not more than 4 ft., may be subsequently tightened when the concrete is packed as tightly as possible by hand, and this will serve to consolidate the mass, but the timbering must be very strong to stand this strain and the bolts must all be screwed up simultaneously, they and the washers upon which the

nuts turn being well greased. Where only one side of the wall is exposed as a retaining-wall, screw or hydraulic jacks may be used to effect the same purpose. The only thing is to find a suitable abutment for them. This may be a temporary pile well strutted. The jack bears on this and the boarding which is forced up against the work by a number of jacks simultaneously. Iron plates are much more satisfactory than timber for this purpose. In the case of a bridge pier the jacks may butt against the abutments by means of long struts, but these struts must necessarily be of a big section to comply with Formulæ 154 and 155, the value of W being either the pressure shown by the gauge of a hydraulic jack, or that calculated for the screw jack when the pitch of the screw is known, and the length of the lever, a man being supposed capable of exerting a force at the end of the lever of 120 lbs.

NOTE.—Take a bottle jack with a $\frac{1}{4}$-in. pitch square thread, and a lever 2 ft. long from centre of screw. At this 2 ft. radius 100 lbs. is applied. In one revolution the end of the handle traverses a distance $= 2 \times 3{\cdot}1416 = 6{\cdot}2832$ ft. $\times 100 = 628{\cdot}32$ lb. feet for every revolution. But in one revolution the screw only moves $\frac{1}{4}$ in. $= \frac{1}{48}$ ft. $\therefore 628{\cdot}32 \div \frac{1}{48} = 628{\cdot}32 \times 24 = 15{,}079$ lbs. = nearly seven tons pressure exerted.

Care must be taken that the abutment will stand this without damage.

Grouting.—For restoring old masonry bridges and other engineering works grouting is a most important remedy, and one which may, if judiciously carried out, save many a fine structure, as was the case with the Auld Brig of Ayr. Grout is a rich mixture of sand and Portland cement in the form of a watery paste, forced into the structure in such a way that it will fill all the cavities and joints. Grouting must, however, under no circumstances be used to avoid good and careful workmanship in new structures, the solidity of which should be effected by handwork alone. The grout used should be as stiff as possible, conducive with its penetrating every part of the work necessary to repair. Loose rock foundations may be made sound by its aid; in fact, it is often a very wise precaution to take, because rock foundations are not always as sound as general opinion holds them out to be, and loose ground can

even be made solid by its aid. In the case of shafts and tunnels, grout forced in behind the masonry (or cast-iron) lining is very effectual in making the work sound. The grouting machine consists of an iron cylinder which acts as a receiver or reservoir, and by means of pumps one can be forced under compressure up to 100 lbs. per square inch. The air receiver is connected by a flexible tube to the second portion of the apparatus, called the grouting-pan. This consists of another cylinder furnished with a handle and spindle, to which are attached arms or beaters, so as to convert it into a churn, the object being to keep the cement and water and sometimes sand which are introduced through the manhole at the top in motion, so as to prevent sedimentation and separation. The materials are, of course, placed in the cylinder in proper proportions to give the consistency of cream, and the lid then screwed down. This done, the contents are ready to be forced into the crack, the mouth of which on either side of the wall has been clayed up to prevent the grout escaping. Compressed air is then admitted to the grouting-pan and as soon as the necessary valve is opened the contents are discharged into the wall.

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